



Seismic Evaluation – Phase 2: Report

Sonoma County – Chanate Hospital

3325 Chanate Drive
Santa Rosa, CA

ZFA Project: 14565

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EXECUTIVE SUMMARY

The Chanate Hospital campus, located at 3325 Chanate Road in Santa Rosa, CA, has been reviewed for **Life Safety** performance level using the ASCE 41-13 Standard for Seismic Evaluation and Retrofit of Existing Buildings, Tier 1 and Tier 2 Evaluations. The buildings were reviewed using the original construction documents, structural Tier 1 checklists, and site visits. Non-structural elements were not included in the scope of this review. Items indicated as non-compliant by Tier 1 checklists were reviewed using Tier 2 evaluation procedures.

See attached chart at end of Executive Summary for catalogue of reviewed structures including date of construction, square footage, number of stories and structural system type. Also included are the assumed previous Occupancy and Risk Categories for reviewed structures (per current code, 2013 CBC). Structures could be occupied for usages that fall within the same or lower Risk/Occupancy categories without requiring updates for current code compliance. All assumed Occupancy and Risk Categories noted are preliminary and should be verified by the County of Sonoma.

A cost estimate has been prepared by Leland Saylor Associates for the reviewed buildings on this campus and is presented in Appendix H relating to the structural recommendations noted within this report. The work represented is to be considered a reasonable order of magnitude cost estimate to retrofit the deficiencies identified in this initial evaluation. Further analysis and actual retrofit design drawings would refine the accuracy of the required work and subsequent cost estimate. The proposed construction would result in a safer and more resilient building improving performance during a seismic event by reducing the loss-of-life risk and the extent and cost of required repairs. This objective aligns with the performance objective of **Life Safety** per the scope of this report. As noted in the report, the potential fault rupture at or under the structure remains as risk even if the retrofit work is completed. Therefore, the retrofit cost estimate does not reflect remediation of this risk. As requested by the client, the cost estimate was prepared to include an annual escalation rate of 5.0% with the assumption that the mid-point of construction will occur at the one-year mark. Additional modification factors and allowances included are as indicated within the cost estimate. See matrix at end of Executive Summary indicating repair types and cost estimates for each building.

The structural review resulted in the following structural and geotechnical findings and recommendations for improvement at each building in order of significance:

Building 1 (1999-2004 Cath Labs)

Structural

- No “non-compliant structural” items were found during the Tier 1 review of Building 1. No Tier 2 checks were required.

Geotechnical

- **Surface Fault Rupture:** Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly define and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. Based on size of building, existing seismic separations between smaller portions of the building, and redundancy of wood construction, minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing and non-structural (MEP and architectural framing); however, the structure overall is not likely to collapse. Significant fault rupture within the building envelope is likely to damage the building beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

Occupancy Group and Risk Category

- Based on the previous occupancy group L (laboratory) and risk category III, the current building code allows all risk category III and lower occupancies within this structure per CBC Table 1604.5 without triggering current code compliance.

Cost Estimate

- A cost estimate has been prepared by Leland Saylor Associates for this building and is presented in Appendix H relating to the structural recommendations noted within this report. The intent of the cost estimate shown for building 1 reflects the reasonable order of magnitude estimate for full replacement and relocation of building 1 to a location outside the fault rupture zone. As noted in the report, all building 1 additions are a bench mark building and no other seismic improvements were noted except to mitigate the fault rupture location. The proposed construction would result in a safer and more resilient building improving performance during a seismic event by reducing the loss-of-life risk and the extent and cost of required repairs. Building 1, which includes the 1999, 2001 and 2004 portions, was the only building where replacement was an option. Therefore the cost estimate reflects an optional full replacement and relocation outside the fault rupture zone with an estimated construction cost of \$4,888,488.00 for informational purposes only.

Building 2 (1972 Acute Care Hospital)

Structural

- Steel frame moment connections are not adequate to resist Tier 1 or Tier 2 calculated seismic demands acting on the structure. While some specific checks on moment frame components may be determined to be adequate, other components are determined to not be adequate and the connection as a whole is considered not adequate. Regardless of adequacy, all moment frame connections are considered “Pre-1994 Northridge Earthquake” moment connections which historically have poor performance in a major earthquake. Without adequate retrofit of these connections, the frames could fail by means of brittle fracture of some or all of the connections, which results in loss of lateral capacity. Frames with lost capacity have the potential to drift excessively, cause significant damage, and continually weaken during shaking leading to a potential collapse once full capacity is lost especially in a large, long duration, seismic event, or a shorter duration large event with many large aftershocks. Shorter events may experience more localized fractures at the joints and associated damage requiring repair to restore strength to the lateral system of the structure. Strengthening of the moment frame connections by reducing the beam cross section, adding flange and column cover plates as sketched in Appendix G is recommended. *Structural Priority: High*
- Steel braced frames in the penthouse are not adequate to resist Tier 1 or Tier 2 calculated seismic demands acting on this portion of the structure. Additionally, only one brace is present on each elevation, which results in no redundancy and complete reliance upon compression buckling. Modern design methodology provides opposing braces that improve redundancy and

- places half of the loads in compression and tension. Addition of a second brace is recommended. The existing brace configuration should be replaced or supplemented with a stiffer element to resist buckling of the brace itself. Without adequate retrofit, the frames could fail and cause significant damage within the penthouse. This does not represent a major life safety concern as the penthouse contains various utility equipment and is not intended for occupancy. Strengthening of the existing brace and addition of supplemental braces as sketched in Appendix G is recommended. *Structural Priority: High*
- Adjacent building structures do not meet the minimum required clear separation to subject building for independent seismic performance. Additional analysis may be performed to estimate horizontal movement in a seismic event. Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause life safety structural concerns within the subject building. Further analysis of possible egress issues is recommended. *Structural Priority: Low*

Geotechnical

- **SURFACE FAULT RUPTURE:** Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. Based on size of building, existing seismic separations between smaller portions of the building and minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing and non-structural (MEP and architectural framing); however, the structure overall is not likely to collapse. Significant fault rupture within the building envelope is likely to damage the building beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

Occupancy Group and Risk Category

- Based on the previous building usages, per the 2013 CBC it can be categorized as occupancy group I-2 (Hospital) and risk category IV. The current building code allows all risk category IV and lower occupancies within this structure per CBC Table 1604.5 without triggering current code compliance.

Cost Estimate

- A cost estimate has been prepared by Leland Saylor Associates for this building is presented in Appendix H relating to the structural recommendations noted within this report. The work represented is to be considered a reasonable order of magnitude cost estimate to retrofit the

deficiencies identified in this initial evaluation. Further analysis and actual retrofit design drawings would refine the accuracy of the required work and subsequent cost estimate. The proposed construction would result in a safer and more resilient building improving performance during a seismic event by reducing the loss-of-life risk and the extent and cost of required repairs. This objective aligns with the performance objective of **Life Safety** per the scope of this report. As noted in the report, the potential fault rupture at or under the structure remains as risk even if the retrofit work is completed. Therefore, the retrofit cost estimate does not reflect remediation of this risk. Building 2, the 1972 portion, cost estimate reflects retrofit scope of steel moment frame members and connections, and retrofit of the penthouse lateral steel bracing system with an estimated construction cost of \$2,838,477.

Buildings 3-6 (1956 Hospital Wing)

Structural

All Buildings:

- The adjacent 2-story structures do not meet the minimum Tier 1 required clear separations for independent seismic performance. Additional more detailed Tier 3 analysis must be performed to approximate horizontal movement of each structure during a seismic event including the strengthening effects of retrofit options provided in this report. The current scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3). Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause life safety structural concerns within the subject building. Egress issues are recommended to be further analyzed. See additional comments and recommendations pertaining to Building 6 and Building 3 Steel appendages below.

Structural Priority: Low

Buildings 3, 4 and 5:

- Stirrups in concrete beams over means of egress do not have proper hook configurations. Jacketing beams with fiber reinforced polymer (FRP) strips or steel plates is recommended as means of controlling localized damage from a seismic event and adding ductility/resiliency to this 'fuse' type member.

Structural Priority: Low

Building 6:

- The steel ledger connecting the concrete roof slab of Building 6 to Building 5 is the critical connection in the seismic performance of building 6. The connection utilizes archaic expansion anchors to transfer in-plane and out-of-plane forces to the Building 5 lateral system that have no reliable tensile capacity. It is recommended a new steel ledger be welded to the existing ledger and attached to the Building 5 concrete walls with adhesive anchors meeting current code requirements.

Structural Priority: Medium

- Dowels to the foundation stem wall below the concrete masonry unit (CMU) wall piers were not specified in the existing drawings or visible during site review. Because the wall piers have a large height-to-width ratio, flexural capacity is important to the seismic performance of the walls. The flexural capacity is dependent on the ability of the boundary steel to transfer loads to the foundation through dowels. Recommend selective demolition to identify wall dowels for further analysis or concrete infill of spandrel/window bays to reduce flexural seismic demands.

Structural Priority: Low

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Buildings 3 Steel Appendages:

- There are four (4) steel framed appendages adjacent to Building 3 which have a minimum of two (2) bays of moment frames in both directions and are directly connected to the exterior concrete walls of Building 3. The majority of these moment frame members and connections, as well as the roof and floor connections to the main concrete structure, are not adequate to resist Tier 1 or Tier 2 calculated seismic force demands and prescriptive requirements. These moment frames are considered “Pre-1994 Northridge Earthquake” which historically have poor performance in a major earthquake. In addition, the stiff concrete shear walls in plane with the flexible steel moment frames do not have compatible stiffness. Most of the seismic forces generated from the mass of the steel appendages will be transferred through the ledger connections and braced by the stiffer concrete shear walls. Tier 2 analysis shows that the heavily reinforced concrete shear wall structure is adequate to resist the increase in seismic mass from the four (4) appendages, but the existing ledger connections between the structures are insufficient. Without adequate retrofit, these ledger connections could fail and cause damage to the steel structures and represent a hazard to occupants. If seismic load is shifted to the moment frames once the ledger connection has failed, the moment connections are susceptible to brittle fracture and loss of lateral capacity. Retrofit recommendations include the following three (3) options:
 - a. Strengthen roof and floor ledger connections between appendages and Building 3. Add new stiffer lateral system (steel plate shear wall or brace frame) at the exterior wall of the appendages parallel to adjacent existing wall to more closely match lateral stiffness of concrete shear wall system and reduce drift on steel structure.
 - b. Separate steel appendages from Building 3. Remove existing ledgers, cut back roof/floor decking, add additional steel gravity framing to support deck edges, and install compressible expansion material and top cover plates between floors at wall openings. Strengthen the moment frame beams, columns, and connections.
 - c. Completely remove steel appendages.

See Appendix G for plans and details specifying retrofit options.

Structural Priority: Medium

Geotechnical

- **SURFACE FAULT RUPTURE:** Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. As shown in the Surface Fault Rupture Map in Appendix B, portions of these buildings (in particular Building #3) are located directly over the projections of the fault traces as determined by previous geologic surveys. Based on size of building, existing seismic separations between the buildings, and redundancy and ductility of the reinforced concrete structure, minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing and non-structural (MEP and architectural framing); however, the structure overall is not

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likely to collapse. Fault rupture within the building envelope is likely to damage the building beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

Occupancy Group and Risk Category

- Based on the previous buildings' usages, per the 2013 CBC they can be categorized as occupancy group I-2 (Hospital) and risk category IV. The current building code allows all risk category IV and lower occupancies within these structures per CBC Table 1604.5 without triggering current code compliance.

Cost Estimate

- A cost estimate has been prepared by Leland Saylor Associates for this building and is presented in Appendix H relating to the structural recommendations noted within this report. The work represented is to be considered a reasonable order of magnitude cost estimate to retrofit the deficiencies identified in this initial evaluation. Further analysis and actual retrofit design drawings would refine the accuracy of the required work and subsequent cost estimate. The proposed construction would result in a safer and more resilient building improving performance during a seismic event by reducing the loss-of-life risk and the extent and cost of required repairs. This objective aligns with the performance objective of **Life Safety** per the scope of this report. As noted in the report, the potential fault rupture at or under the structure remains as risk even if the retrofit work is completed. Therefore, the retrofit cost estimate does not reflect remediation of this risk. Total estimated construction cost of retrofit scope for this report is \$318,426 broken down between buildings per below:
 - a. Cost estimates for Buildings 3-5, the 1956 concrete structures, reflect a retrofit scope of jacketing concrete beams over means of egress with "FRP" or steel plates with an estimated construction cost of \$47,489.
 - b. Cost estimates for Building 6, the 1961 Emergency Room portion, reflect a retrofit scope of concrete wall infills and strengthening of ledger connections with an estimated construction cost of \$96,222.
 - c. Cost estimates for the four (4) Building 3 Steel Appendages, built roughly in 1988, reflect a retrofit scope (Option A) of adding new steel brace frames or steel plate shear wall and strengthening roof and floor ledger connections with an estimated construction cost of \$174,715. As an alternative to Option A, Options B and C are also presented with associated construction cost of \$256,142 and \$133,448 respectively.

Building 7 (1936 Original Hospital Building)

The 1936 building is the oldest and most ornate building on the Chanate Hospital campus, and thus may have the greatest historic value. The recommendations for this building largely focus on the two primary options of complete demolition or complete retrofit, but neither option may be feasible for historic preservation purposes or budgetary purposes. Therefore, further study may be required to determine which areas of the building have the most historic interest and potential functionality to be retained for a partial retrofit and/or partial demolition option.

Structural

- The lateral force resisting system, consisting of diagonal rod braced wall panels, lacks load path and is severely deficient. Load path issues include, but are not limited to:
 - Chords and collectors are neither well-defined nor detailed for seismic forces at the roof diaphragms. Minimal structural continuity exists, consisting of thin gage metal tracks with long unbraced lateral lengths and minimal splices.

- Chords and collectors are neither well-defined nor detailed for seismic forces at the floor diaphragms. Minimal partial structural continuity exists, consisting of small steel ledger angles that are discontinuous at re-entrant corners and changes in framing direction.
- Floor diaphragms are discontinuous at interior stud walls, which run full height of the building, and do not have a shear transfer load path across the interior diaphragm gaps.
- Shear transfer from the roof diaphragm to the braced wall panels is neither well-defined nor detailed for seismic forces. Minimal lateral connection consists of weak-axis bending of thin gage metal roof joist supports.
- Shear transfer from the second floor diaphragm to the braced wall panels is neither well-defined nor detailed for seismic forces. Minimal lateral connection consists of weak axis bending of thin gage metal stud webs.
- Shear transfer from the first floor diaphragms to the foundation stem walls is neither well-defined nor detailed for seismic forces. Minimal lateral connection consists of weak axis bending of thin gage metal studs.
- Development of the braced wall panel rod ends is neither well-defined nor detailed for seismic forces. Minimal connections rely upon eccentric force concentrations applied to the face of thin gage metal stud webs.
- Transfer of the wall panel overturning forces from edge studs is neither well-defined nor detailed for seismic forces. Minimal connection consists of eccentric loading and weak axis bending of thin gage metal tracks.

Even if all load path issues are resolved by means of retrofit, the diagonal rods and wall panel end studs are substantially deficient when comparing design force demand and capacity at Tier 1 force levels. The building has a high probability of severe damage or catastrophic collapse during a large seismic event. Considering that the building does not provide a substantial lateral force resisting system, one of the following options is recommended:

- 1) Demolish the building.
- 2) Provide an entirely new lateral force resisting system within the building, consisting of structural steel braced frames, structural steel chords and collectors, and metal stud blocking at all diaphragm discontinuities. The construction impacts for this retrofit are intensive. See Appendix G – Strengthening Sketches.
- 3) Selectively demolish portions of the building and retrofit the remaining areas that are chosen to be kept for program functionality or historical value purposes.
- 4) Abandon the building and provide adequate barrier to limit access or proximity to the building on all sides. Adjacent buildings 3 and 8 should not rely upon Building 7 for egress nor should egress be allowed within proximity of Building 7 due to potential collapse. Adjacent buildings and covered walkways should also be strengthened as required to withstand impact due to potential collapse of Building 7.

Structural Priority: High

- The gravity load system, consisting of concrete slabs over steel open web joists at 32" on center, supported by light gage metal stud walls and concrete basement walls, has various corrosion, deterioration, and damage issues. If a new lateral force resisting system is provided as optionally recommended above, then the following is recommended for repair:
 - The 6" wide bearing wall studs are typically corroded at the base, and in some cases, are cut or bent. The metal stud sill tracks that attach the bearing walls to the concrete basement walls are in poor condition, with widespread corrosion throughout the building. All bearing walls throughout the building are recommended to be surveyed at the basement level for damage and deterioration. All sill tracks with advanced corrosion are recommended to be replaced and new 16" long stud sections are recommended to be spliced to the bottom of the existing studs that have damage or advanced corrosion. For estimation purposes, it should be assumed that approximately 25% of the stud walls require repair at the basement.

- The 2½" concrete slab at the first floor has many locations with areas of spalling and with corroded reinforcement at the bottom of slab. The entire elevated first floor slab is recommended to be surveyed for damage and deterioration. All slab locations with excessive spalling or advanced corrosion are recommended to be supported with angles at 12" on center that span between floor joists. For estimation purposes, it should be assumed that less than 5% of the first floor area requires repair.
- The steel open web joists at the first floor have many locations where chords have been cut or damaged. All floor joists at the first floor are recommended to be surveyed for damage and deterioration. All damaged chords are recommended to be spliced with new angles. For estimation purposes, it should be assumed that less than 5% of the first floor joists require repair.
- The 8" thick concrete basement walls have several locations that have been saw-cut without proper header reinforcement. All basement walls are recommended to be surveyed for un-reinforced saw-cut openings, and are recommended to be strengthened with steel channel headers. For estimation purposes, it should be assumed that three openings require reinforcement.

Structural Priority: High

- Adjacent Building 3 and the adjacent covered walkways do not meet the minimum Tier 1 required clear separation for independent seismic performance. The 12" clear gap at adjacent Building 8 does meet the minimum Tier 1 requirement. However, by observation, the clear separation at all locations will be insufficient to protect Buildings 3 and 8 and the covered walkways from damage due potential collapse of Building 7 if the building is abandoned as optionally recommended above. If a new lateral force resisting system is provided as optionally recommended above, then the clear separations are recommended to be analyzed for adequacy. However, remediation of this deficiency may not be feasible.

Structural Priority: Low

Geotechnical

- SURFACE FAULT RUPTURE: Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. Based on the substantial deficiencies of the building system as discussed in the structural summary above, fault offsets are likely to exacerbate the already large collapse potential. If a new lateral force resisting system is provided as optionally recommended, significant fault rupture within the building envelope could still likely damage the building beyond repair or future use depending on the magnitude of the offset. Thus, a comprehensive geotechnical review would be prudent to determine if retrofit is warranted.

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An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

Structural Priority: High

Occupancy Group and Risk Category

- Based on the previous building usages, per the 2013 CBC it can be categorized as occupancy group I-2 (Institutional) and risk category IV. The current building code allows all risk category IV and lower occupancies within this structure per CBC Table 1604.5 without triggering current code compliance.

Cost Estimate

- A cost estimate has been prepared by Leland Saylor Associates for this building and is presented in Appendix H relating to the structural recommendations noted within this report. The work represented is to be considered a reasonable order of magnitude cost estimate to retrofit the deficiencies identified in this initial evaluation. Further analysis and actual retrofit design drawings would refine the accuracy of the required work and subsequent cost estimate. The proposed construction would result in a safer and more resilient building improving performance during a seismic event by reducing the loss-of-life risk and the extent and cost of required repairs. This objective aligns with the performance objective of **Life Safety** per the scope of this report. As noted in the report, the potential fault rupture at or under the structure remains as risk even if the retrofit work is completed. Therefore, the retrofit cost estimate does not reflect remediation of this risk. Building 7, the “1936” building, cost estimate reflects the Option 2 Full Retrofit scope, including addition of a new lateral force resisting system and repair of the existing gravity system corrosion and damage, with an estimated construction cost of \$10,999,281. Option 1 Demolition is also presented with associated construction cost of \$1,140,510. Option 3, a combination of retrofit and demolition, construction cost will fall somewhere between, depending upon the chosen scope of work.

Building 8 (1956 Kitchen/Storage Building)

Structural

- Anchorage connections between the longitudinal concrete walls and the roof diaphragm are not adequate to resist Tier 1 or Tier 2 calculated seismic demand forces acting perpendicular to the wall plane. Without adequate anchorage of walls to diaphragms, the walls could potentially pull away from the roof diaphragm and become a collapse hazard. Strengthening of the wall anchorage is recommended. *Structural Priority: High*
- The diagonal sheathed diaphragms are not adequate to resist Tier 1 and Tier 2 calculated seismic demand forces. Without an adequate diaphragm the building cannot support the heavy concrete walls for seismic demand acting perpendicular to the walls which could cause a potential collapse hazard. *Structural Priority: High*
- The separation between the covered walkway and adjacent 1936 building is not adequate to prevent the pounding or interaction between the structures during a seismic event, causing localized minor damage to the covered walkway. Damage due to this condition is not anticipated to cause life safety structural concerns within the subject building though localized damage will occur. Reference the separate evaluation report for the adjacent 1936 structure for potential damage and recommendations for that structure. Egress issues are recommended to be further analyzed. *Structural Priority: Low*

Geotechnical

- Surface Fault Rupture: Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault,

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to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. However, this particular building is the only campus building reviewed that is entirely located in the area 'less likely' to be subject to fault rupture (see geotechnical map and summary). Based on size and orientation of the building, and redundancy of the systems minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing and non-structural (MEP and architectural framing); however, the structure overall is not likely to collapse. Significant fault rupture within the building envelope is likely to damage the building beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

Occupancy Group and Risk Category

- Based on the previous building usages, per the 2013 CBC it can be categorized as risk category II. The current building code allows all risk category II and lower occupancies within this structure per CBC Table 1604.5 without triggering current code compliance, increasing the risk category would trigger current code compliance and potentially significant retrofit and modifications to the structure.

Cost Estimate

- A cost estimate has been prepared by Leland Saylor Associates for this building is presented in Appendix H relating to the structural recommendations noted within this report. The work represented is to be considered a reasonable order of magnitude cost estimate to retrofit the deficiencies identified in this initial evaluation. Further analysis and actual retrofit design drawings would refine the accuracy of the required work and subsequent cost estimate. The proposed construction would result in a safer and more resilient building improving performance during a seismic event by reducing the loss-of-life risk and the extent and cost of required repairs. This objective aligns with the performance objective of **Life Safety** per the scope of this report. As noted in the report, the potential fault rupture at or under the structure remains as risk even if the retrofit work is completed. Therefore, the retrofit cost estimate does not reflect remediation of this risk. Building 8, the Kitchen and Storage portion, cost estimate reflects the diaphragm and wall anchorage strengthening retrofit scope, including reroofing with an estimated construction cost of \$457,467.

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Building 9 (1987 Ambulance Canopy)

Structural

- Adjacent structures do not meet the minimum Tier 1 required clear separation to the subject canopy for independent seismic performance. Additional analysis may be performed to estimate horizontal movement in a seismic event. Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause life safety structural concerns within the subject canopy. Further analysis of possible egress issues is recommended. Possible remediation of the hazard could be to install knee braces between the columns and beams above head clearance level to stiffen the canopy structure reducing expected deflections in a seismic event (see schematic retrofit detail **SSK-1**).
Structural Priority: Low

Geotechnical

- Surface Fault Rupture: Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Based on the relative small size and value of this structure, relocation of the building is not likely a reasonable solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. Based on the canopy's small size and seismic weight, existing seismic separations between adjacent buildings and the flexibility of cantilevered column systems, minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing; however, the structure overall is not likely to collapse. Significant fault rupture within the canopy envelope is likely to damage the canopy beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

Occupancy Group and Risk Category

- Based on the previous building usages, per the 2013 CBC it can be categorized as occupancy group I-2 (Institutional Group: Hospitals with emergency treatment facilities) and risk category IV. The current building code allows all risk category IV and lower occupancies within this structure per CBC Table 1604.5 without triggering current code compliance.

Cost Estimate

- A cost estimate has been prepared by Leland Saylor Associates for this building and is presented in Appendix H relating to the structural recommendations noted within this report. The work

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represented is to be considered a reasonable order of magnitude cost estimate to retrofit the deficiencies identified in this initial evaluation. Further analysis and actual retrofit design drawings would refine the accuracy of the required work and subsequent cost estimate. The proposed construction would result in a safer and more resilient building improving performance during a seismic event by reducing the loss-of-life risk and the extent and cost of required repairs. This objective aligns with the performance objective of **Life Safety** per the scope of this report. As noted in the report, the potential fault rupture at or under the structure remains a risk even if the retrofit work is completed. Therefore, the retrofit cost estimate does not reflect remediation of this risk. Total estimated construction cost of retrofit scope for this report is \$12,566 reflecting a retrofit scope of twelve tube steel knee braces.

The following evaluation report details our findings.

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Building Summary TableCHANATE HOSPITAL BUILDING CRITERIA SUMMARY TABLE¹

CRITERIA	1999-2004 CATH LAB BUILDING 1	1972 ACUTE CARE HOSPITAL BUILDING 2	ADDITION BUILDING 3	EAST WING BUILDING 4	WEST WING BUILDING 5	ER ADDITION BUILDING 6	1936 ORIGINAL HOSPITAL BUILDING 7	1956 KITCHEN/ STORAGE BUILDING 8	1987 AMBULANCE CANOPY BUILDING 9
STORIES	1	4	2	1	1	1	2 + BASEMENT	1	1
AREA (SF)	8,037	56,181	34,742 (TOTAL)	(INCLUDED IN BUILDING 3)	(INCLUDED IN BUILDING 3)	(INCLUDED IN BUILDING 3)	38,017 + BASEMENT	8,600	1,500
ASCE 41-13 BUILDING SYSTEM TYPE	W2 – WOOD FRAMED STRUCTURES	S1 – STEEL MOMENT FRAME WITH STIFF DIAPHRAGMS	C2 – CONCRETE SHEAR WALL WITH STIFF DIAPHRAGMS	C2 – CONCRETE SHEAR WALL WITH STIFF DIAPHRAGMS	C2 – CONCRETE SHEAR WALL WITH STIFF DIAPHRAGMS	RM2 – REINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS	LIGHT GAUGE METAL FRAMING WITH STEEL ROD BRACING (S2 USED FOR BRACES)	C2A – CONCRETE SHEAR WALL WITH FLEXIBLE DIAPHRAGMS	S1A – CANTILEVERED STEEL COLUMNS
ASCE 41-13 BENCHMARK YEAR (UBC)	1976	1994	1994	1994	1994	1994	N/A	1994	1994
ASCE 41-13 BENCHMARK BUILDING?	YES	NO	NO	NO	NO	NO	NO	NO	NO
PREVIOUS ASSUMED OCCUPANCY (2013 CBC)	L	I-2	I-2	I-2	I-2	I-2	I-2	B	I-2
RISK CATEGORY (2013 CBC TABLE 1604.5)	III	IV	IV	IV	IV	IV	IV	II	IV

¹ SEE ATTACHED APPENDIX A FOR SCHEMATIC CAMPUS SITE MAP

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Cost Estimate Summary Table

CHANATE HOSPITAL BUILDING DEFICIENCY/RETROFIT COST SUMMARY TABLE									
DEFICIENCY ¹	1999-2004 CATH LAB BUILDING 1	1972 ACUTE CARE HOSPITAL BUILDING 2	ADDITION BUILDING 3	1956 HOSPITAL WINGS EAST WING BUILDING 4	WEST WING BUILDING 5	ER ADDITION BUILDING 6	1936 ORIGINAL HOSPITAL BUILDING 7	1956 KITCHEN/ STORAGE BUILDING 8	1987 AMBULANCE CANOPY BUILDING 9
SURFACE FAULT RUPTURE	✓	✓	✓	✓	✓	✓	✓	✓	✓
LOAD PATH							✓		
WALL ANCHORAGE						✓		✓	
STEEL BRACE FRAMES							✓		✓
STEEL MOMENT FRAMES		✓	✓						
CONCRETE BEAMS OVER EXITS			✓	✓	✓				
DIAPHRAGMS							✓	✓	
WALLS						✓			
ADJACENT BUILDINGS		✓	✓	✓	✓	✓	✓	✓	✓
ESTIMATED RETROFIT COST ²	\$0	\$2,838,477	\$190,546	\$10,533	\$21,125	\$96,222	\$10,999,281	\$457,467	\$12,566
1956 HOSPITAL WINGS TOTAL = \$318,426									

¹ DEFICIENCIES LISTED HIGHEST TO LOWEST IN PROJECTED ORDER OF SIGNIFICANCE
² ESTIMATED RETROFIT COSTS DO NOT ADDRESS SURFACE FAULT RUPTURE OR ADJACENT BUILDING DEFICIENCY

INTRODUCTION

The purpose of this evaluation is to review and evaluate the structural systems of the subject building against criteria provided by ASCE 41-13. The evaluation criteria have been tailored for specific building types and desired levels of building performance. This standard is based on criteria developed from observation of structural and non-structural damage occurring in previous earthquakes and means to identify general deficiencies based on anticipated behavior of specific building types.

The evaluation begins with a Screening Phase (Tier 1) to assess primary components and connections in the seismic force resisting system through the use of standard checklists and simplified structural calculations. Checklist items are general in nature and intended to highlight building components that do not exceed conventional construction guidelines. If the element is compliant, it is anticipated to perform adequately under seismic loading without additional review or strengthening. Items indicated as non-compliant in a Tier 1 checklist are considered potential deficiencies that require further analysis.

A limited, deficiency-based Evaluation Phase (Tier 2) can then be used to review the items determined to be potential deficiencies by Tier 1 checklists and simplified calculations. Non-compliant items are evaluated for calculated linear seismic demand as determined by ASCE 41-13. If the elements are compliant per Tier 2 analysis, the Tier 1 deficiency is waived. However, if the element remains non-compliant after the more detailed Tier 2 analysis, repair or remediation of deficiency is recommended.

In certain cases, a more detailed Systematic Evaluation (Tier 3) may be more appropriate for complex structures where a Tier 2 analysis may be considered significantly conservative. A Tier 3 structural evaluation generally requires a substantially greater level of effort than a Tier 2 review.

Structural Performance Objective

Per ASCE 41-13, a structural performance objective consists of a target performance level for structural elements in combination with a specific seismic hazard level. For seismic assessment of the subject building, the Basic Performance Objective for Existing Buildings (BPOE) was selected. While the BPOE seeks safety for occupants with reasonable confidence, it allows existing structures to be reviewed for less than current code loading with the understanding that the cost savings from not retrofitting the subject building up to current code standards may result in greater repair costs in event of an earthquake. Buildings meeting the BPOE are expected to experience nominal damage from relatively frequent, moderate earthquakes, but have the potential for significant damage and economic loss from the most severe, though less frequent, seismic events.

For the purposes of this review to the BPOE, for this a building of this occupancy category (as described by ASCE 7) the desired level of performance is **Life Safety (3-C)** for this non-essential structure. The Life Safety Performance Level as described by ASCE/SEI 41-13: *‘Structural Performance Level S-3 is defined as the post-earthquake damage state in which a structure has damaged components but retains a margin against the onset of partial or total collapse. Non-Structural Performance Level N-C is the post-earthquake damage state in which Nonstructural Components may be damaged, but the consequential damage does not pose a Life Safety threat.’*

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SITE OVERVIEWGeneral Site Description

The Chanate campus is located on a gently sloped lot approximately 1.1 miles east of Highway 101 off of Chanate Rd in Santa Rosa, CA. The campus was formerly known as the County Hospital and until recently was occupied by Sutter Medical Health Center.

Site Seismicity (Earthquake Activity)

Per ASCE 41-13, 'Seismicity', or the potential for ground motion, is classified into regions defined as Low, Moderate, or High. These regions are based upon mapped site accelerations S_s and S_1 which are then modified by site coefficients F_a and F_v to produce the Design Spectral Accelerations, S_{DS} (short period) and S_{D1} (1-second period). The successful performance of buildings in areas of high seismicity depends on a combination of strength, ductility of structural components, and the presence of a fully interconnected, balanced, and complete lateral force resisting system. Where buildings occur in lower levels of seismicity, the strength and ductility required for successful performance is significantly reduced, and building components or connections with additional strength capacity can in some cases be adequate despite lacking ductility.

Based on the geotechnical report provided for the subject site, the soil profile of this building can be classified as Site Class C per ASCE 41-13 for use in determination of site coefficients F_a and F_v .

Per the site values indicated by USGS data and evaluated using earthquake load equations and tables of ASCE 41-13, the site is located in a region of **High Seismicity** with a design short-period spectral response acceleration parameter (S_{DS}) of 1.656g and a design spectral response acceleration parameter at a one second period (S_{D1}) of 0.892g (approximate values for entire campus, individual building calculations use building specific site response parameters). Both of these parameters exceed the lower boundaries for high seismicity classification, 0.5g for S_{DS} and 0.2g for S_{D1} .

<i>Level of Seismicity*</i>	S_{DS}	S_{D1}
Low	< 0.167g	< 0.067g
Moderate	≥ 0.167g < 0.500g	≥ 0.067g < 0.200g
High	≥ 0.500g	≥ 0.200g

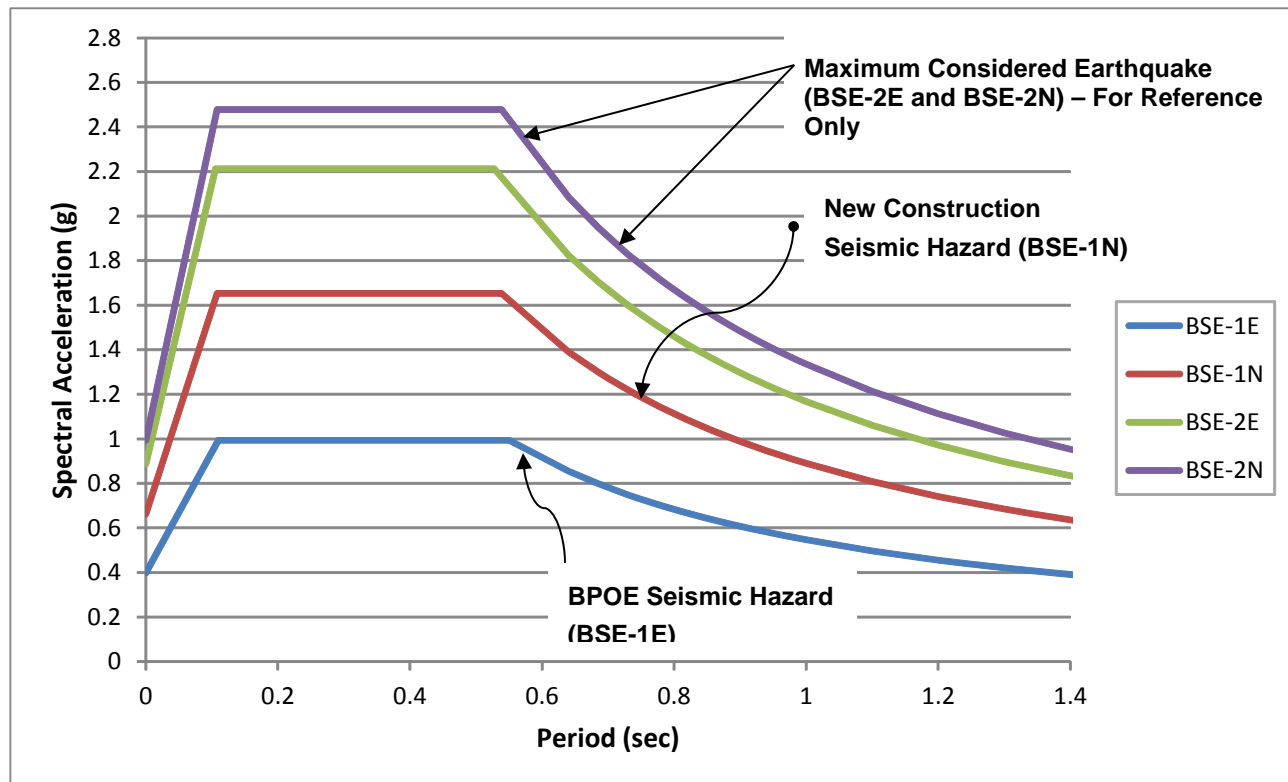
*Where S_{DS} and S_{D1} values fall in different levels of seismicity, the higher level shall be used.

The spectral response parameters S_s and S_1 for review of the subject building were obtained for the BSE-1E seismic hazard level for existing structures (BPOE). The acceleration values were adjusted for the maximum direction and site class in accordance with ASCE 41 Section 2.4.1, and compared to BSE-1N (used by current building code for design of new buildings) to determine the design values for the Tier 1 and Tier 2 analyses, since values obtained for the BSE-1E hazard level need not exceed the hazard levels for new construction.

The following charts depict the response spectra for the multiple seismic hazard levels defined by ASE 41-13, two existing hazard levels and two hazard levels corresponding to code design of new structures (ASCE 7). Note that the seismic hazard level for design of existing structures is nearly equal to that for new construction.

Seismic Hazard Level*	Building Code Reference	Peak Spectral Acceleration S_a
BSE-1E	ASCE 41-13 (20%/50yr)	0.99g
BSE-1N	ASCE 7-10 Design Basis Earthquake (DBE)	1.65g
BSE-2E	ASCE 41-13 (5%/50yr)	2.21g
BSE-2N	ASCE 7-10 Maximum Considered Earthquake (MCE)	2.48g

* Seismic hazard levels denoted with 'E' for existing buildings or 'N' for new building equivalency.



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BUILDING 1 (1999-2004 Cath Labs)

Evaluation Overview

This seismic evaluation report for the existing building located at 3325 Chanate Road in Santa Rosa, CA, is based on the following:

- The American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI 41-13) Standard for Seismic Evaluation and Retrofit of Existing Buildings - Tier 1 and Tier 2 (non-compliant items only), Life Safety level structural evaluation criteria.
- Two site visits for general review of structures performed on 11/05/14 and 11/06/14. No destructive testing or removal of finishes was performed or included in scope.
- Review of following original drawings:
 - 1999 Modular Cath Lab structural drawings by GV Custom Modular Construction, Inc (dated 1999)
 - 2001 MRI Addition structural drawings by DASSE Design Inc. Structural Engineers (dated 2000)
 - 2004 Cath Lab Addition structural drawings by MKM & Associates (dated 2004)
- Review of following geotechnical reports and hazard maps:
 - Site geologic Hazard investigations and project specific geotechnical reports as indicated in the Geotechnical Summary.
 - Liquefaction Susceptibility and Surface Fault Rupture hazard maps from Association of Bay Area Governments (ABAG).
- Review of non-structural elements is not included.

Structural System and Materials Description

General

Building 1 is composed of (3) smaller buildings separated by seismic gaps and built in 3 separate phases as follows:

- The 1999 Modular Cath Lab (Building 1a): Designed in 1999 is a 4916 square foot single story wood framed structure built with modular units. The building has a rectangular footprint of 58'-0" wide x 95'-0" long. A covered pedestrian walkway connects the 1999 Cath Lab to Building 2 (1972 Addition) and is seismically isolated from adjacent structures. This covered walkway structure was not reviewed, however it was noted the covered walkway is a steel structure with (4) cantilevered steel column lateral systems. Quick checks were performed on the walkway columns based on Tier 1 and flexural stresses appear to be compliant. Additionally, the walkway was also noted to be a benchmark structure for an S1a type building since it was designed after the 1994 UBC provisions.
- The 2001 MRI Addition (Building 1b): Designed in 2001 is an 1138 square foot single story wood framed structure. The building has a rectangular footprint of 23'-9" wide x 48'-0" long.
- The 2004 Cath Lab Addition (Building 1c): Designed in 2004 is an 1862 square foot single story wood framed structure. The building has a rectangular footprint of 37'-4" wide x 48'-0" long.

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Roof Framing

- The 1999 Modular Cath Lab (Building 1a): 8" deep light gage metal roof joists spaced at 24" on center with wood structural sheathing roof diaphragm.
- The 2001 MRI Addition (Building 1b): Wood I-joist at 16" on center with wood structural sheathing roof diaphragm.
- The 2004 Cath Lab Addition (Building 1c): Wood I-joist at 24" on center with wood structural sheathing roof diaphragm.

First Floor structure

- The 1999 Modular Cath Lab (Building 1a): 12" deep light gage metal joists over crawl space
- The 2001 MRI Addition (Building 1b): Concrete Slab on Grade
- The 2004 Cath Lab Addition (Building 1c): Concrete Slab on Grade

Walls

All buildings are built with wood stud walls sheathed with wood structural panels.

Lateral Force Resisting System

The vertical lateral force resisting system for all buildings is wood stud walls sheathed with wood structural panels.

Foundations

Foundations for all buildings consist of shallow concrete spread footings and/or isolated pad footings for interior columns.

Field Verification and Condition Assessment

The structures on campus appear in generally good structural condition with minimal structural damage or deterioration apparent, and appear to be constructed in general accordance with the provided structural drawings.

Building Type

Per ASCE/SEI 41-13, this building can be classified as **Building Type W2: Wood Frames, Commercial and Industrial**. As described by ASCE/SEI 41-13: *'These buildings are commercial or industrial buildings with a floor area of 5,000 square feet or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. The foundation system may consist of a variety of elements. Seismic forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, oriented strand board, stucco, plaster, straight or diagonal wood sheathing, or braced with rod bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.'*

Historical Performance

Modern wood frame structures detailed to resist seismic loads were generally not built prior to 1934, except for public schools in high seismic areas. In general, seismic provisions for wood framed structures started to be incorporated into building codes in the 1950's. After 1970, well-defined lateral-force-resisting systems were usually incorporated as part of the design in high seismic areas. Seismic performance of these types of structures is dependent on proper detailing and quality of construction. Wood framed structures with diagonal lumber or plywood sheathed shear wall systems have demonstrated adequate performance in past earthquakes provided they had low height-to-length aspect ratios, acted as a unit, had an adequate number of shear walls, and were reasonably symmetric in plan and elevation. In particular, plywood shear wall systems with height-to-length aspect ratios less than 2-to-1 typically provide acceptable earthquake load resistance strength. However,

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plywood shear walls generally require hold-downs at each end to resist overturning especially in multistory structures.

Benchmark Buildings

In addition to classifying buildings by type of construction, ASCE 41 identifies 'Benchmark Buildings' for each type. The detailing of seismic force resisting systems in Benchmark Buildings is generally considered to meet the performance requirements of ASCE 41. When a building is determined to meet Benchmark Building requirements through field verification of construction compliant with benchmark code requirements, only review of foundation and non-structural elements is required. Even though a building appears to meet the benchmark criteria, a full analysis may still be recommended under certain circumstances.

For building type W2 with Life Safety structural performance level, the 1976 UBC seismic design provisions are referenced as the oldest permitted standard. Since, the subject buildings were constructed during or after 1999, and per the provided documentation were constructed under the 1997 CBC code, it meets the criteria of a Benchmark Building, and does not require further analysis. However, at the request of the client, a complete Tier 1 analysis of the building was performed.

Findings and Recommendations

The ASCE 41-13 Tier 1 Basic Life Safety and Building Type Specific Checklists indicate the primary building structure as non-compliant in one (1) area for Life Safety Performance.

- a. SURFACE FAULT RUPTURE (ASCE Section A.6.1.3) – *“Surface fault rupture and surface displacement at the building site are not anticipated.”* The site is located in the Alquist-Priolo special study zone per the California Division of Mines and Geology (CDMG) Santa Rosa Quadrangle Map published in 1983. Multiple geologic hazard evaluations and geotechnical reports completed by five geotechnical firms were reviewed for this site. The oldest reviewed report was completed in 1978 while the most recent was completed in 2002. The exact location of faults in the area of the Chanate Hospital buildings is not clearly determined by the reports due to differing data and conclusions. The 2002 report was performed by Rutherford & Chekene (R&C) and included a geologic hazard evaluation by Gilpin Geosciences. Gilpin Geosciences Report summarizes the surface fault rupture hazard:

“Based on the preponderance of lineaments and other fault-related features observed by Gilpin Geosciences and others in the site vicinity, along with the lack of clear resolution of differing interpretations of onsite and offsite geologic structures, we conservatively judge the overall potential for fault rupture at the site to be high. There may exist areas within the site that are sufficiently free of active faults so as to allow future construction of structures for human occupancy.”

A lineament is a feature in a landscape which is an expression of an underlying geologic structure such as a fault. The 1978 Cooper Clark & Associates report estimated the maximum potential offset at the surface as 25 inches horizontally and 2.5 inches vertically. The report summarized the potential surface rupture behavior:

“If surface rupture were to occur along the part of the fault near the hospital, the displacement could occur along the relatively well located trace mapped to the west, along the approximately located trace mapped near this site, or along other traces, such as those found nearby in our trenches – a zone probably 980 feet wide.”

Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred. Four of the five geotechnical engineers that provided information for the site concluded there were likely fault

traces or fault related features extending through at least some portion of the site. Rutherford & Chekene's (R&C) conclusions for the site varied. In the 1986 report R&C concluded there were fault traces on the site but they are not considered active. Reports completed between 1987 and 1992 concluded there were no fault traces at the specific reviewed locations and the entire health care facility was suitable for development from a geologic/geotechnical viewpoint. The final R&C report in 2002 concluded it was likely that fault traces projected through the site and under the existing Medical Center complex. Additionally the report stated there was a high potential for surface rupture. There does appear to be areas where fault traces may be less likely to occur based on the fault maps provided. See the geotechnical summary for a complete discussion of all reviewed information pertaining to surface fault rupture.

The 1978 Cooper Clark & Associates report was the most extensive study of potential fault traces on the site that was reviewed. R&C performed many investigations with differing conclusions however the latest R&C report reviewed stated the overall potential for fault rupture at the site to be high. This finding is in line with Cooper Clark, Herzog and HLA (Herzog and HLA reports are for buildings not included in the scope of work for this project). The following is a summary of potential fault traces as it affects the building. See the Geotechnical Summary for a complete summary of all reviewed reports.

Two of trenches for the 1978 Cooper Clark & Associates evaluation were approximately located along the north and south edges of where building 1 is located. In both trenches fault traces were found that project below the building. This building was built approximately 20 years later so an additional geotechnical investigation was completed that may have addressed the potential for fault rupture in more detail for the building location but those report were not provided for review. The 1986 R&C report found several fault traces that approximately aligned in location and orientation with the Cooper Clark report that would project beneath this building; however they classified the faults as non-active (more than 11,000 years old). The 2002 R&C report stated it is prudent to assume that active faults may project under the existing Medical Center complex.

RECOMMENDATION: Based on the information contained in the reviewed reports and the CDMG maps, the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly define and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is a possible solution for this structure, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. Based on size of building, existing seismic separations between smaller portions of the building, and redundancy of wood construction, minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing and non-structural (MEP and architectural framing); however, the structure overall is not likely to collapse. Significant fault rupture within the building envelope is likely to damage the building beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

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BUILDING 2 (1972 Acute Care Hospital)

Evaluation Overview

This seismic evaluation report for the existing building located at 3325 Chanate Road in Santa Rosa, CA, is based on the following:

- The American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI 41-13) Standard for Seismic Evaluation and Retrofit of Existing Buildings - Tier 1 and Tier 2 (non-compliant items only), Life Safety level structural evaluation criteria.
- Two site visits for general review of the structure performed on 11/5/14 and 11/6/14. No destructive testing or removal of finishes was performed or included in scope.
- Review of following original drawings:
 - Structural drawings by John E Brown & Associates and Graham & Hayes Structural Engineers (1970).
 - Project Book Specifications (Volume I) (May 1, 1970)
- Review of a previous Seismic Hazard Investigation of the building performed by H.J. Degenkolb and Associates in August 1979
- Existing material properties as indicated in Appendix D.
- Review of following geotechnical reports and hazard maps:
 - Site geologic Hazard investigations and project specific geotechnical reports as indicated in the Geotechnical Summary.
 - Liquefaction Susceptibility and Surface Fault Rupture hazard maps from Association of Bay Area Governments (ABAG).
- Review of non-structural elements is not included in this scope. Some deficient items have been previously noted by others (see Senate Bill 1953 section below for discussion, however these items relate to the previous use as a state hospital facility)

Structural System and Materials Description

General

Building 2 was designed in 1970 and built in 1972. Building 2 consists of 4 stories of nearly symmetrical rectangular levels and a roof penthouse containing much of the building's mechanical equipment. There are 2 small appendages for the elevators and stairs at the northwest and southwest of the building. At the lower levels, these appendages also connect to other parts of the campus, a 1-story connector at the north end to building 1, and a 2-story connector on the south end to building 3. Both connectors have seismic separations at the interface to the adjoining structures. There is a cantilevered entry canopy at the west side of the building. The total combined building footprint is approximately 14,000 square feet. Building 2 is toward the northern side of the subject campus (See Appendix B – Schematic Site Map).

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Roof Framing

The structure is approximately 61'-4" tall (including penthouse) with the top of the main roof occurring at approximately 48'-10" above the first floor (ground) level. The penthouse roof structure is steel framed consisting of 4" concrete over 3" metal deck over steel wide flange beams. Steel beams are supported by steel wide flange columns.

Fourth and Third Floor Framing

The fourth and third floor structures consist of 3" concrete over 3" metal deck over steel wide flange beams. Steel beams are supported by steel wide flange columns. Column splices occur approximately 1'-7" above the fourth floor.

Second Floor Framing

The second floor structure consists of 3" concrete over 3" metal deck over steel wide flange beams. Steel beams are supported by steel wide flange columns.

Walls

Typical exterior walls are 6 inch thick metal studs spaced at 16"oc. Brick veneer is anchored to the metal studs @ 12"oc vertically, located at each stud. A #9 wire runs horizontally through the brick joints located at each anchor location (12"oc vertically). At lintels, the brick veneer sits on a steel Tee section connected to the main steel framing. In some locations the exterior finish is stucco. The metal studs form a parapet at the roof level that ranges from 1 to 4 feet above the roof diaphragm.

Lateral Force Resisting System

The primary lateral force resisting system for the combined structure is a 4-level, 6-bay perimeter steel moment frame consisting of composite wide flange beams and columns. At the penthouse roof structure, a diagonally braced frame occurs in 1 bay on each of the four perimeter elevations, consisting of double channel members. The floor and roof diaphragms are concrete over metal deck that is welded to the moment frame beams and can be considered a stiff/rigid diaphragm.

Foundations

The ground floor is a 4" thick concrete slab-on-grade with wire mesh reinforcing at mid-depth. The interior foundations are isolated pad concrete footings of various size and depth embedded in to sub-grade. A 4-foot wide (6-foot wide at the north elevation) continuous reinforced concrete spread footing supports the exterior stud walls and perimeter moment frame columns and moment frames of the building and connects the walls, moment frames, foundation and slab together. The steel columns are securely pinned to the substantial continuous footings with (4) 2-1/2" diameter rods embedded 4'-6" with plate washers.

Field Verification and Condition Assessment

The building 2 structure appears in generally good structural condition with minimal structural damage or deterioration apparent, and appears to be constructed in general accordance with the provided structural drawings. Destructive demolition could not be performed at this time and access to the ceiling space was not available due to asbestos concerns. Steel framing around the elevator shaft appeared to have spray-applied fireproofing covering the steel connections as well as concrete encased steel so visual observation of the steel connections could not be performed at this time.

Material Properties

Basic properties for existing structural materials found on existing building documentation, through testing or ASCE 41 code prescribed minimum structural values utilized in the analysis calculations can be found in Appendix D.

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Building Type

Per ASCE/SEI 41-13, this building can be classified as **Building Type S1: Steel Moment Frames With Stiff Diaphragms**. As described by ASCE/SEI 41-13: *‘These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists, or steel trusses. Seismic forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames. Where the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. The foundation system may consist of a variety of elements.’*

Historical Performance

Modern steel moment frame systems came about in the 1960’s when beam flanges and webs were welded directly to the columns to create fully restrained sections. Shear tabs bolted to the beam webs and welded to the columns later replaced welded beam webs. These welded-flange and bolted-web connections were used extensively from the 1970’s through the early 1990’s and are now known as the pre-Northridge connections. These frames did not perform as well as expected during the 1994 Northridge earthquake. A significant number of the frames inspected after the earthquake exhibited visible cracking in the beam flange-to column welds resulting in brittle failures of the beam to column connection that could cause floors to collapse. In a few rare cases the flanges completely fractured and the damage extended into either the shear tab or column panel zone. Buildings that relied on deep beams that are stronger than the columns are more susceptible to this type of damage. Currently moment frames are designed to force beam yielding away from the column and the connection by using strong columns compared to beams and reducing the beam section adjacent to the connection at columns. This connection allows the beam to yield and prevent brittle failures. Moment frame buildings are generally flexible and subject to large interstory drifts. Their ductility is achieved through yielding of beams and or shear yielding of column panel zones at beam-column connections. This inelastic behavior allows moment frames to sustain many cycles of loading and load reversals (seismic loading). The subject building was designed prior to the 1994 Northridge earthquake and appears rely on a deep beam system with limited redundancy. The frame connections from the beam to the columns are detailed in the standard method for pre-Northridge structures. As with all buildings of this type there is a risk of brittle failure of the frame connections.

Benchmark Buildings

In addition to classifying buildings by type of construction, ASCE 41 identifies ‘Benchmark Buildings’ for each type. The detailing of seismic force resisting systems in Benchmark Buildings is generally considered to meet the performance requirements of ASCE 41. When a building is determined to meet Benchmark Building requirements through field verification of construction compliant with benchmark code requirements, only review of foundation and non-structural elements is required. Even though a building appears to meet the benchmark criteria, a full analysis may still be recommended under certain circumstances.

For building type S1, the 1994 UBC seismic design provisions are referenced as the oldest permitted standard. Since, the subject building was constructed in 1972, and based on the provided documentation assumed constructed under the 1970 UBC code, it does not meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

California Senate Bill 1953

The State of California Senate Bill 1953 (SB 1953) establishes a seismic safety building standards program under the jurisdiction of the Office of Statewide Health Planning and Development (OSHPD) for hospital buildings. The Bill emphasizes that acute care facilities should remain operational after an earthquake. SB 1953 requires and

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defines the procedures to determine the Structural (SPC) and Non-Structural Performance Category (NPC) rating for hospitals. The ratings range from 0 (zero) as the worst to 5 as the best. All acute care hospital facilities must be SPC2 and NPC3 by year 2008 or extend the deadline per SB 1801 to 2013. Furthermore these facilities must achieve SPC3 and NPC5 ratings by year 2030. Acute care facilities that do not achieve these ratings must be taken out of service or used for non-acute care purposes.

OSHPD currently recognizes the 1972 Wing as SPC1 and NPC1. The facility originally self-declared as SPC3 but did not follow through with required documentation and later de-classified to SPC1. Typically any pre-1973 building without a retrofit can only be SPC1 or SPC2. The moment frame conditions discussed further in this evaluation would typically preclude a rating higher than SPC1. Additionally, the facility submitted a NPC2 compliance program report to OSHPD but the work was never completed or documented. The NPC2 compliance program report documented the bulk medical gas system, fire alarm system, emergency lighting and means of egress system, and two paging cabinets and the Ambulance-Hospital relay elements of the communication system as deficient.

Findings and Recommendations

The ASCE 41-13 Tier 1 Basic Life Safety and Building Type Specific Checklists indicate the primary building structure as non-compliant in five (5) areas for Life Safety Performance.

- a. ADJACENT BUILDINGS (ASCE Section 4.3.1.2) – *“The clear distance between the building being evaluated and any adjacent building shall be greater than 4 percent of the height of the shorter building for Life Safety and Immediate Occupancy.”* Single story connection wing to building 1 to the north of building 2 is constructed with approximately 4” clear distance to the subject structure. Proximity to adjacent structure is less than the required 6” (4 percent of 12.5’ concourse height), and is non-compliant. Similarly, the 2 story connection wing to building 3 to the south of building 2 is constructed with approximately 4” clear distance to the subject structure. Proximity to adjacent structure is less than the required 11.5” (4 percent of 24’ concourse height), and is non-compliant.

RECOMMENDATION: Additional analysis may be performed to estimate horizontal movement in a seismic event. Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause life safety structural concerns within the subject building. Egress issues are recommended to be further analyzed.

- b. FLEXURAL STRESS CHECK (ASCE Section A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2) – *“The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9, is less than F_y . Columns need not be checked if the strong column–weak beam checklist item is compliant.”* Approximately 40% of the members of the North-South frames and 37% of the members of the East-West frames do not comply with the Tier 1 and Tier 2 evaluation. The average demand/capacity ratio (DCR) using the Average Maximum stress, is approximately $1.16 > 1.0$.

RECOMMENDATION: Retrofit the beams and columns to add additional plates welded to the flanges and/or webs increasing the strength of the members. Completing a full analysis of the structure through Tier 3 may reduce the required retrofit. The current scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3).

- c. MOMENT-RESISTING CONNECTIONS (ASCE Section A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1) – *“All moment connections are able to develop the strength of the adjoining members based on 110% of the expected yield stress of steel per AISC 341, Section A3.2.”* The majority of beam to

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column connections at the main structure are non-compliant in both directions per the Tier 1 and Tier 2 evaluation.

RECOMMENDATION: Retrofit the beam to column connections by adding cover plates to the top and bottom beam flanges at the column interface. This will deliver the full capacity of the beam section to the column. Completing a full analysis of the structure through Tier 3 may reduce the required retrofit. The current scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3).

- d. STRONG COLUMN—WEAK BEAM (ASCE Section A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5) – “*The percentage of strong column—weak beam joints in each story of each line of moment frames is greater than 50%*”. The beam to column moment ratios in both directions are non-compliant in both directions per the Tier 1 and Tier 2 evaluation.

RECOMMENDATION: Retrofit the beam to column connections in both directions by adding a reduced beam section “dog bone” cut (RBS) in the beam flanges away from the column face, forcing the rotational plastic hinge condition to occur in the beam as dictated by modern codes. This effectively makes the columns stronger than the beams (Strong Column – Weak Beam theory). It should be noted that adding RBS cuts in beams can increase building drifts by approximately 10% which may lead to ASCE 41-13 drift check deficiencies which are currently compliant, however the other recommended moment frame retrofit work would help compensate for the increased drift due to the RBS cuts in the beams. Completing a full analysis of the structure through Tier 3 may reduce the required retrofit. The current scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3).

- b. SURFACE FAULT RUPTURE (ASCE Section A.6.1.3) – “*Surface fault rupture and surface displacement at the building site are not anticipated.*” The site is located in the Alquist-Priolo special study zone per the California Division of Mines and Geology (CDMG) Santa Rosa Quadrangle Map published in 1983. Multiple geologic hazard evaluations and geotechnical reports completed by five geotechnical firms were reviewed for this site. The oldest reviewed report was completed in 1978 while the most recent was completed in 2002. The exact location of faults in the area of the Chanate Hospital buildings is not clearly determined by the reports due to differing data and conclusions. The 2002 report was performed by Rutherford & Chekene (R&C) and included a geologic hazard evaluation by Gilpin Geosciences. Gilpin Geosciences Report summarizes the surface fault rupture hazard:

“Based on the preponderance of lineaments and other fault-related features observed by Gilpin Geosciences and others in the site vicinity, along with the lack of clear resolution of differing interpretations of onsite and offsite geologic structures, we conservatively judge the overall potential for fault rupture at the site to be high. There may exist areas within the site that are sufficiently free of active faults so as to allow future construction of structures for human occupancy.”

A lineament is a feature in a landscape which is an expression of an underlying geologic structure such as a fault. The 1978 Cooper Clark & Associates report estimated the maximum potential offset at the surface as 25 inches horizontally and 2.5 inches vertically. The report summarized the potential surface rupture behavior:

“If surface rupture were to occur along the part of the fault near the hospital, the displacement could occur along the relatively well located trace mapped to the west, along the approximately located trace mapped near this site, or along other traces, such as those found nearby in our trenches – a zone probably 980 feet wide.”

Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly define and located outside of the Santa Rosa area, the fault location at the hospital site is inferred. Four of the five

geotechnical engineers that provided information for the site concluded there were likely fault traces or fault related features extending through at least some portion of the site. Rutherford & Chekene's (R&C) conclusions for the site varied. In the 1986 report R&C concluded there were fault traces on the site but they are not considered active. Reports completed between 1987 and 1992 concluded there were no fault traces at the specific reviewed locations and the entire health care facility was suitable for development from a geologic/geotechnical viewpoint. The final R&C report in 2002 concluded it was likely that fault traces projected through the site and under the existing Medical Center complex. Additionally the report stated there was a high potential for surface rupture. There does appear to be areas where fault traces may be less likely to occur based on the fault maps provided. See the geotechnical summary for a complete discussion of all reviewed information pertaining to surface fault rupture.

The 1978 Cooper Clark & Associates report was the most extensive study of potential fault traces on the site that was reviewed. R&C performed many investigations with differing conclusions however the latest R&C report reviewed stated the overall potential for fault rupture at the site to be high. This finding is in line with Cooper Clark, Herzog and HLA (Herzog and HLA reports are for buildings not included in the scope of work for this project). The following is a summary of potential fault traces as it affects the building. See the Geotechnical Summary for a complete summary of all reviewed reports.

The 1978 Cooper Clark & Associates report projected two fault traces to extend below the building. The 1986 R&C report found several fault traces that approximately aligned in location and orientation with the Cooper Clark report that would project beneath this building; however they classified the faults as non-active (more than 11,000 years old). The 2002 R&C report stated it is prudent to assume that active faults may project under the existing Medical Center complex.

RECOMMENDATION: Based on the information contained in the reviewed reports and the CDMG maps, the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. Based on size of building, existing seismic separations between smaller portions of the building and minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing and non-structural (MEP and architectural framing); however, the structure overall is not likely to collapse. Significant fault rupture within the building envelope is likely to damage the building beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

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Non-Structural

Non-Structural items were not evaluated at this time. Previous evaluations of this building have noted some deficient items (see Senate Bill 1953 section above for discussion, however these items relate to the previous use as a state hospital facility). Additionally, a previous Seismic Hazard Investigation of the building was performed by H.J. Degenkolb and Associates in August 1979. Excerpt items not related to specific hospital use are noted as follows:

The partitions are typically metal stud with gypsum board with many extending from the floor to structure above. Ceilings are typically of acoustical tile or gypsum board. Although the ceiling and partition system may not comply with the current State requirements for new hospital construction, we believe the numerous small rooms will provide reasonable bracing to the system. If fault displacement occurs beneath the building, there will be considerable racking of partitions and ceiling and some will certainly collapse. A light metal furring is spanned across two ceiling support channels and two small bolts clamp the channel and light fixture together. Although this detail should be reasonable under many seismic exposures, we envision the failure of this detail in many cases if the building is racked due to fault displacement beneath the building. A preferred method would involve individual safety or support wires from each fixture to the structure above, which is difficult with surface mounted fixtures and perhaps impossible to achieve within the crowded ceiling space.

The building's mechanical systems were briefly reviewed, both in the 1970 addition (noted as the 1972 Building 2 in this ZFA report) and in the new boiler house (not included in this ZFA report). Most of the equipment observed has not been bolted to the floor or restrained for seismic motions and sliding can be expected. An example of a compressor unit in the penthouse of the 1970 addition utilizes vibration isolators which appear to be of a brittle cast type that failed in the San Fernando earthquake of 1971. It should be noted, however, that the sliding or movement of this mechanical equipment represents a minor hazard to personnel as few people are ever present in the areas containing the equipment. The elevator system utilizes counterweights and pulleys. We do not believe that these elevators comply with the new seismic requirements for elevators and counterweights which have been adopted since the 1971 San Fernando earthquake.

Cost Estimate

A cost estimate has been prepared by Leland Saylor Associates for this building is presented in Appendix H relating to the structural recommendations noted within this report. The work represented is to be considered a reasonable order of magnitude cost estimate to retrofit the deficiencies identified in this initial evaluation. Further analysis and actual retrofit design drawings would refine the accuracy of the required work and subsequent cost estimate. The proposed construction would result in a safer and more resilient building improving performance during a seismic event by reducing the loss-of-life risk and the extent and cost of required repairs. This objective aligns with the performance objective of **Life Safety** per the scope of this report. As noted in the report, the potential fault rupture at or under the structure remains as risk even if the retrofit work is completed. Therefore, the retrofit cost estimate does not reflect remediation of this risk. Building 2, the 1972 portion, cost estimate reflects retrofit scope of steel moment frame members and connections, and retrofit of the penthouse lateral steel bracing system with an estimated construction cost of \$2,838,477.

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BUILDINGS 3-6 (1956 Hospital Wing)

Evaluation Overview

This seismic evaluation report for the existing building located at 3325 Chanate Road in Santa Rosa, CA, is based on the following:

- The American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI 41-13) Standard for Seismic Evaluation and Retrofit of Existing Buildings - Tier 1 and Tier 2 (non-compliant items only), Life Safety level structural evaluation criteria.
- Two site visits for general review of the structure performed on 11/7/14 and 11/20/14. No destructive testing or removal of finishes was performed or included in scope.
- Review of following original drawings and report:
 - Buildings 3,4, and 5 structural drawings by Art B. Smith, Structural Engineer (1953)
 - Building 6 ER Addition structural drawing by Edwin A. Verner, Structural Engineer (1961)
 - Building 3 Steel Appendages structural drawings by MKM & Associates (1987)
 - "Seismic Hazard Investigation, Community Hospital of Sonoma County, Santa Rosa, California" by H.J. Degenkolb & associates, Engineers, dated 08/04/78
- Existing material properties from original drawings as indicated in Appendix D.
- Review of following geotechnical reports and hazard maps:
 - Site geologic Hazard investigations and project specific geotechnical reports as indicated in the Geotechnical Summary.
 - Liquefaction Susceptibility and Surface Fault Rupture hazard maps from Association of Bay Area Governments (ABAG).
- Review of non-structural elements is not included.

Structural System and Materials Description

1956 Concrete Structures (Buildings 3, 4 & 5):

General

Buildings 3, 4 and 5 are concrete structures designed in 1953 and built in 1956. Building 3 consists of a long rectangular 2-story structure with another 2-story square wing projecting to the east. Four (4) 2-story, steel framed bathroom appendages were added later adjacent to Building 3 and are discussed in more detail below. The total combined Building 3 footprint is approximately 22,000 square feet. Building 3 is surrounded by buildings on all four sides. To the west of Building 3 is Building 4 which is a similar yet smaller 1-story concrete rectangular structure. The total Building 4 footprint is approximately 2500 square feet. To the east is Building 5 which is also a similar yet smaller 1-story "L-shaped" concrete structure. A long narrow concrete masonry structure (Building 6) was later added to the south of Building 5 and is discussed in more detail below. The total Building 5 footprint is approximately 8400 square feet. Building 2, a 4-story steel structure built in 1972, is located to the north of Building 3 and Building 7, the old 2-story structure built in 1936 abuts Building 3 to the south. All interfaces

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between Building 3 and adjoining structures are separated by 8 inch wide seismic gaps. Buildings 3, 4, and 5 are near the center of the subject campus (See Appendix B – Schematic Site Map).

Roof Framing

The main roof structure of Building 3 occurs approximately 22'-8" above the first floor (ground) level. The roof structure consists of a 4.5 inch thick concrete slab reinforced typically with a #3 bottom rebar mat spaced at 12 inches on center with #4 top rebar spaced at 12 inches on center typically over interior concrete beam supports. The roof structure of Building 4 is similar to Building 3 except it occurs 11'-4" above ground level. Slab to exterior walls dowels are #4 top bars spaced at 12 inches on center which are embedded and hooked into the walls. Bottom mat rebar is developed 8 inches into the exterior walls and hooked 180 degrees at the ends. The roof concrete slab spans a maximum of 16 feet between concrete beams typically 14 inches wide and 18 inches deep. Concrete beams are supported typically by 16 inch square concrete columns which are reinforced typically with four vertical #8 or #9 bars and #3 stirrups spaced at 12 inches on center.

Floor Framing

The main floor structure of Building 3 occurs approximately 11'-4" above the first floor (ground) level. The floor structure consists of a 6.5 inch thick concrete slab reinforced typically with a #4 bottom rebar mat spaced at 16 inches on center with #4 top rebar spaced at 12 inches on center typically over interior concrete beam supports. The roof structure of Building 5, which was originally designed as a future floor, consists of the same slab thickness and general rebar layout as the floor structure of Building 3. Slab to exterior walls dowels are #4 top bars spaced at 12 inches on center which are embedded and hooked into the walls. Bottom mat rebar is developed 8 inches into the exterior walls and hooked 180 degrees at the ends. The roof concrete slab spans a maximum of 16 feet between concrete beams typically 16 inch wide by 22 inch deep. Concrete beams are supported typically by 16 inch square concrete columns which are reinforced typically with four vertical #8 or #9 bars and #3 stirrups spaced at 12 inches on center.

Walls

Typical exterior concrete walls are 10 inches thick with #4 rebar spaced at 16 inches on center each face in both horizontal and vertical directions. Above and below all walls openings are (2) #6 horizontal continuous bars. Wall vertical rebar is doweled and hooked into continuous wall footings.

Lateral Force Resisting System

The primary lateral force resisting system for the combined structure is concrete shear wall. The floor and roof diaphragms are concrete slabs and can be considered a stiff/rigid diaphragm.

Foundations

Foundations are typically continuous 30 inch wide by 12 inch deep continuous concrete footings. Interior concrete square pad footings occur at concrete columns.

Field Verification and Condition Assessment

The structures on campus appear in generally good structural condition with minimal structural damage or deterioration apparent, and appear to be constructed in general accordance with the provided structural drawings. Destructive demolition could not be performed and access to the ceiling space was limited due to asbestos concerns. Minor, non-structural cracking was observed at various locations in the concrete slab-on-grade and plaster ceiling. Parallel cracking was observed in the slab-on-grade and first floor ceiling at the north-west re-entrant corner of Building 3; however, no cracking was visible in the concrete floor slab above at this location.

Material Properties

Basic properties for existing structural materials found on existing building documentation utilized in the analysis calculations can be found in Appendix D.

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1961 Concrete Masonry Structure (Building 6):

General

Building 6 is a 12 foot wide one story Emergency Room addition to the south side of Building 5 and is approximately 105' long and 1,260 square feet. Only the south concrete masonry unit (CMU) wall faces the exterior as the north and west sides are directly connected to Building 5 and the east end has an interior seismic separation to Building 3. A small Telecommunications room was added sometime after 1961 as an appendage to the southwest corner of Building 6. There were no existing drawings reviewed for this Telecommunications room but it is an approximately 250 square foot rectangle CMU box with one door opening to the exterior. The roof slabs are approximately 12 feet above the floor slab and 14 feet above adjacent grade.

Roof Framing

The roof of the structure is a 5 inch concrete slab sloping toward the interior connection to the Building 5 concrete shear walls. The roof slab is reinforced with transverse #4 bars at 7 inches on center and #3 bars at 18 inches on center in the longitudinal direction. The roof slab is connected to the Building 5 walls with a continuous L6x4x½ ledger using 7/8"Ø machine bolts dry packed at 3 feet on center. The machine bolts utilize archaic expansion nut units with little to no tensile capacity. The roof slab is supported on the southern exterior CMU wall with a standard keyed bearing connection and dowels for out of plane anchorage. The roof slab spans transversely (north-south) and does not rely on end walls for vertical support.

Floor Framing

The floor of Building 6 is a 4 inch concrete slab reinforced with welded wire mesh. The slab is supported on 6 inches of rock over compacted fill creating a slab level with Building 5 elevated approximately 22 inches above the adjacent grade. The slab is doweled into both the existing Building 5 and exterior CMU wall concrete stem walls.

Walls

The south wall of the Building 6 ER Addition is constructed of ten 55 inch wide reinforced CMU wall piers interconnected with precast concrete spandrels. The north wall is the existing Building 5 concrete shear wall line. The Telecommunications Addition walls are open to the north interior, and constructed of CMU on the other three exterior sides. The east facing wall of the Telecommunications addition is CMU doweled into the top of an existing concrete site wall.

Lateral Force Resisting System

In the transverse (north-south) direction the Building 6 addition is intended to be tied into the Building 5 (concrete shear wall) lateral system through the roof slab steel ledger with archaic expansion anchors that have minimal reliable tensile capacity. In the longitudinal direction, the roof slab is tied into Building 5 through the steel ledger and machine bolts in shear on the north side. The south side utilizes reinforced CMU wall piers with unknown dowels into the foundation to resist seismic forces. The Telecommunications addition is tied into the Building 6 lateral system on the north end and is supported by CMU shear walls on all other sides.

Foundations

The exterior walls of the ER Addition are supported on 10 inch concrete stem walls with #4 bars at 12 inches on center extending down to a 14 inch wide by 12 in deep continuous footing reinforced with one #5 bar. The foundations for the Telecommunications addition were not observed but are assumed to be of similar construction.

Field Verification and Condition Assessment

The CMU walls appear in generally good structural condition with minimal structural damage or deterioration apparent. The concrete roof slab is in moderate condition with various areas of concrete spalling to the underside

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of the slab. The spalling appears to be the result of localized concrete breakout from post-installed ceiling tie rod anchorage. The critical steel ledger connection to Building 5 is in poor condition showing significant signs of rust. The roof slab slopes down to this connection and the roof waterproofing has apparently failed in some areas. The steel ledger has water damage with visible rust stains against the concrete wall below the member. The structure appears to be constructed in general accordance with the provided structural drawings; however, dowels from the CMU wall piers were not specified and were not observed inside the wall. As these dowels are critical to the performance of the CMU wall piers in a seismic event, it is recommended they be verified in field by selective demo and/or pachometer testing/scanning as needed for more extensive evaluation.

Material Properties

Basic properties for existing structural materials found in ASCE 41 code prescribed minimum structural values and on the existing building documentation utilized in the analysis calculations can be found in Appendix D.

1987 Steel Structures (Buildings 3 Appendages):

General

The four (4) steel framed appendages to Building 3 were designed in 1987 and estimated construction was in 1988. These 2-story rectangular appendages are dispersed around and connected directly to the exterior concrete walls of Building 3 and were designed to increase the size of the bathrooms for the hospital rooms in those particular areas. The roof structures occur approximately 22'-8" above ground level (to match the existing concrete roof elevation). The floor structures occur approximately 11'-4" above ground level (to match the existing concrete floor elevation). The total combined building footprint of the appendages is approximately 2,100 square feet.

Roof Framing

The roof structures consist of Vercor Type "N" 20 gauge metal decking spanning between steel wide flange beams. Steel beams are supported by steel wide flange columns. The edge of roof deck is welded to a continuous steel channel ledger connected to the concrete wall with 5/8" diameter anchors spaced at 24 inches on center.

Second Floor Framing

The second floor structures consist of a total of 3.25" light weight concrete topping over Vercor Type "N" 18 gauge metal deck spanning over steel wide flange beams. Steel beams are supported by steel wide flange columns. The edge of floor deck is welded to a continuous steel channel ledger connected to the concrete wall with 5/8" diameter shallow expansion anchors spaced at 24 inches on center.

Walls

Typical exterior cladding walls are 6 inch thick metal studs spaced at 16"oc. The metal studs form a parapet at the roof level roughly 2'-8" above the roof diaphragm.

Lateral Force Resisting System

The primary lateral force resisting system for the steel structures are 2-level, 1 to 2 bay steel moment frames consisting of steel wide flange beams and columns. There are a minimum of 2 frames in each direction at each steel rectangular appendage. The roof diaphragms are metal deck welded to the beams and can be considered flexible diaphragms. The floor diaphragms are concrete over metal deck welded to the beams and can be considered stiff/rigid diaphragms.

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Foundations

The ground floors are 5" thick concrete slabs-on-grade with #3 @ 18 inches on center reinforcing each way at mid-depth. The steel columns are supported on deeply embedded isolated pad concrete footings between 3 to 4 feet square and 12 inches thick. The column pad footings are deeply embedded below 14 inch square concrete grade beams and the thickened slab edge. The grade beams have (4) #7 longitudinal bars with #3 stirrups spaced at 12 inches on center and are interconnected each way between moment frame columns. The moment frame columns are anchored below the slab and grade beams to the pad footings with (2) 3/4" diameter J-bolts creating a "fixed" base condition for the frame columns. The slab is doweled into the existing concrete stem wall with #5 dowels spaced at 24 inches on center.

Field Verification and Condition Assessment

The steel appendage structures appear in generally good structural condition with minimal structural damage or deterioration apparent, and appear to be constructed in general accordance with the provided structural drawings. One exception is the "Area B" 2-story steel appendage on the north-east side of Building 3 (see Appendix F for location). Area B is considerably reduced in size from what is detailed in the original drawings. Steel framing was coated with spray-applied fireproofing covering the steel members and connections.

Material Properties

Basic properties for existing structural materials found in ASCE 41 code prescribed minimum structural values utilized in the analysis calculations can be found in Appendix D.

Building Type

1956 Concrete Structures (Buildings 3, 4 & 5):

Per ASCE/SEI 41-13, these buildings can be classified as **Building Type C2: Concrete Shear Walls with Stiff Diaphragms**. As described by ASCE/SEI 41-13: *'These buildings have floor and roof framing that consists of cast-in-place concrete flat slabs and concrete beams. Buildings may also have concrete columns and concrete slabs for the gravity framing. Floors are supported on concrete columns or bearing walls. Seismic forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced but often extend throughout the building. In more recent construction, shear walls occur in isolated locations, are more heavily reinforced and have concrete slabs which are stiff relative to the walls. The foundation system may consist of a variety of elements.'*

1961 Concrete Masonry Structure (Building 6):

Per ASCE/SEI 41-13, this building can be classified as **Building Type RM2: Reinforced Masonry Bearing Walls with Stiff Diaphragms**. As described by ASCE/SEI 41-13: *'These buildings have bearing walls that consist of reinforced brick or concrete block masonry. Seismic forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of concrete slabs and are stiff relative to the walls. The foundation system may consist of a variety of elements.'*

1987 Steel Structures (Buildings 3 Appendages):

Per ASCE/SEI 41-13, the floor levels of these building appendages can be classified as **Building Type S1: Steel Moment Frames with Stiff Diaphragms**. As described by ASCE/SEI 41-13: *'These buildings consist of a frame assembly of steel beams and steel columns. Floor framing consists of metal deck with concrete fill supported on steel beams. Seismic forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames. Where the exterior of the structure is concealed, walls consist of metal panel curtain walls. Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. The foundation system may consist of a variety of elements.'*

Per ASCE/SEI 41-13, the roof levels of these building appendages can be classified as **Building Type S1A: Steel Moment Frames with Flexible Diaphragms**. As described by ASCE/SEI 41-13: *‘These buildings consist of a frame assembly of steel beams and steel columns. Roof framing consists of un-topped metal deck (or with lightweight insulating concrete fill) supported on steel beams. Seismic forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of metal deck with lightweight insulating concrete and are flexible relative to the frames. Where the exterior of the structure is concealed, walls consist of metal panel curtain walls. Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. The foundation system may consist of a variety of elements.’*

Historical Performance

1956 Concrete Structures (Buildings 3, 4 & 5):

Concrete slab roof diaphragm and cast-in-place concrete shear wall systems have traditionally performed relatively well in earthquake events provided adequate shear wall length is maintained without localized stresses in short wall piers and provided there are no significant plan or vertical discontinuities such as a difference in stiffness between floors in a multi-storied structure. Positive wall-to-diaphragm connections are also critical to performance. While older buildings of this type are not always ductile and energy dissipative, they do generally provide very stiff and strong structures. Building damage is rarely attributed to a failure of the concrete diaphragms or walls, but rather to failure in related elements in the load path, such as collectors or connections between diaphragms and vertical elements. In highly redundant buildings with many long walls stresses in shear walls are usually low and the performance level is good.

1961 Concrete Masonry Structure (Building 6 ER Addition):

Quality designed, detailed, and constructed reinforced masonry walls with rigid concrete diaphragms have traditionally performed relatively well in earthquake events provided adequate shear wall length is maintained without localized stresses in short wall piers and provided there are no plan or vertical discontinuities such as a soft story. Positive roof-to-wall connections are also critical to performance. While these types of buildings are not typically ductile and energy dissipative, they do generally provide very stiff and strong structures. Building damage is rarely attributed to a failure of the concrete roof diaphragms or CMU walls, but rather to failure in related elements in the load path, such as collectors or connections between diaphragms and vertical elements. In highly redundant buildings with many walls, such as this structure, stresses in shear walls are usually low and the performance level is good.

1987 Steel Structures (Buildings 3 Appendages):

Modern steel moment frame systems came about in the 1960's when beam flanges and webs were welded directly to the columns to create fully restrained sections. Shear tabs bolted to the beam webs and welded to the columns later replaced welded beam webs. These welded-flange and bolted-web connections were used extensively from the 1970's through the early 1990's and are now known as the pre-Northridge connections. These frames did not perform as well as expected during the 1994 Northridge earthquake. A significant number of the frames inspected after the earthquake exhibited visible cracking in the beam flange-to column welds resulting in brittle failures of the beam to column connection that could cause floors to collapse. In a few rare cases the flanges completely fractured and the damage extended into either the shear tab or column panel zone. Buildings that relied on deep beams that are stronger than the columns are more susceptible to this type of damage. Currently moment frames are designed to force beam yielding away from the column and the connection by using strong columns compared to beams and reducing the beam section adjacent to the connection at columns. This connection allows the beam to yield and prevent brittle failures. Moment frame buildings are generally flexible and subject to large inter-story drifts. Their ductility is achieved through yielding of beams and or shear yielding of column panel zones at beam-column connections. This inelastic behavior allows moment frames to sustain many cycles of loading and load reversals (seismic loading). The subject building was

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designed prior to the 1994 Northridge earthquake and appears to rely on a deep beam system with limited redundancy. The frame connections from the beam to the columns are detailed in the standard method for pre-Northridge structures. As with all buildings of this type there is a risk of brittle failure of the frame connections.

Benchmark Buildings

In addition to classifying buildings by type of construction, ASCE 41 identifies 'Benchmark Buildings' for each type. The detailing of seismic force resisting systems in Benchmark Buildings is generally considered to meet the performance requirements of ASCE 41. When a building is determined to meet Benchmark Building requirements through field verification of construction compliant with benchmark code requirements, only review of foundation and non-structural elements is required. Even though a building appears to meet the benchmark criteria, a full analysis may still be recommended under certain circumstances.

1956 Concrete Structures (Buildings 3, 4 & 5):

For building type C2, the 1994 UBC seismic design provisions are referenced as the oldest permitted standard. Since, the subject building was constructed in 1956, and based on the provided documentation assumed constructed under the 1952 UBC code, it does not meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

1961 Concrete Masonry Structure (Building 6 ER Addition):

For building type RM2, the NEHRP1985 seismic design provisions are referenced as the oldest permitted standard. Since, the subject building was constructed in 1961, and per the provided documentation was constructed under the 1958 UBC, it does not meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

1987 Steel Structures (Buildings 3 Appendages):

For building type S1 and S1A, the 1994 UBC seismic design provisions are referenced as the oldest permitted standard. Since, the subject building was constructed approximately in 1988, and based on the provided documentation constructed under the 1979 UBC code, it does not meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

Findings and Recommendations

All Buildings

The ASCE 41-13 Tier 1 Basic Life Safety Checklist indicates the four (4) structures as non-compliant in two (2) areas for Life Safety Performance.

- a. SURFACE FAULT RUPTURE (ASCE Section A.6.1.3) – *“Surface fault rupture and surface displacement at the building site are not anticipated.”* The site is located in the Alquist-Priolo special study zone per the California Division of Mines and Geology (CDMG) Santa Rosa Quadrangle Map published in 1983. Multiple geologic hazard evaluations and geotechnical reports completed by five geotechnical firms were reviewed for this site. The oldest reviewed report was completed in 1978 while the most recent was completed in 2002. The exact location of faults in the area of the Chanate Hospital buildings is not clearly determined by the reports due to differing data and conclusions. The 2002 report was performed by Rutherford & Chekene (R&C) and included a geologic hazard evaluation by Gilpin Geosciences. Gilpin Geosciences Report summarizes the surface fault rupture hazard:

“Based on the preponderance of lineaments and other fault-related features observed by Gilpin Geosciences and others in the site vicinity, along with the lack of clear resolution of differing interpretations of onsite and offsite geologic structures, we conservatively judge the overall potential for fault rupture at the site to be high. There may exist areas within

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the site that are sufficiently free of active faults so as to allow future construction of structures for human occupancy.”

A lineament is a feature in a landscape which is an expression of an underlying geologic structure such as a fault. The 1978 Cooper Clark & Associates report estimated the maximum potential offset at the surface as 25 inches horizontally and 2.5 inches vertically. The report summarized the potential surface rupture behavior:

“If surface rupture were to occur along the part of the fault near the hospital, the displacement could occur along the relatively well located trace mapped to the west, along the approximately located trace mapped near this site, or along other traces, such as those found nearby in our trenches – a zone probably 980 feet wide.”

Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred. Four of the five geotechnical engineers that provided information for the site concluded there were likely fault traces or fault related features extending through at least some portion of the site. Rutherford & Chekene’s (R&C) conclusions for the site varied. In the 1986 report R&C concluded there were fault traces on the site but they are not considered active. Reports completed between 1987 and 1992 concluded there were no fault traces at the specific reviewed locations and the entire health care facility was suitable for development from a geologic/geotechnical viewpoint. The final R&C report in 2002 concluded it was likely that fault traces projected through the site and under the existing Medical Center complex. Additionally the report stated there was a high potential for surface rupture. There does appear to be areas where fault traces may be less likely to occur based on the fault maps provided. See the geotechnical summary for a complete discussion of all reviewed information pertaining to surface fault rupture.

The 1978 Cooper Clark & Associates report was the most extensive study of potential fault traces on the site that was reviewed. R&C performed many investigations with differing conclusions however the latest R&C report reviewed stated the overall potential for fault rupture at the site to be high. This finding is in line with Cooper Clark, Herzog and HLA (Herzog and HLA reports are for buildings not included in the scope of work for this project). The following is a summary of potential fault traces as it affects the building. See the Geotechnical Summary for a complete summary of all reviewed reports.

The 1978 Cooper Clark & Associates report projected two fault traces in the direction of the 1956 building. The 1986 R&C report found several fault traces that approximately aligned in location and orientation with the Cooper Clark report that would project beneath this building; however they classified the faults as non-active (more than 11,000 years old). The 2002 R&C report stated it is prudent to assume that active faults may project under the existing Medical Center complex. During site review for this report the north-south seismic joints in this building appeared to be offset approximately a ½ inch with the building to the west of the joint appearing to have shifted to the north. This offset matches the direction of the fault movement. This may be evidence of fault creep on the site.

RECOMMENDATION: Based on the information contained in the reviewed reports and the CDMG maps, the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would

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include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. As shown in the Surface Fault Rupture Map in Appendix B, portions of these buildings (in particular Building #3) are located directly over the projections of the fault traces as determined by previous geologic surveys. Based on size of building, existing seismic separations between the buildings, and redundancy of the reinforced concrete structure, minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing and non-structural (MEP and architectural framing); however, the structure overall is not likely to collapse. Significant fault rupture within the building envelope is likely to damage the building beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

ADJACENT BUILDINGS (ASCE Section A2.1.2) – *“The clear distance between the building being evaluated and any adjacent building shall be greater than 4 percent of the height of the shorter building for Life Safety and Immediate Occupancy.”* The connection wing to the 4-story Building 2 to the north of Building 3 is constructed with approximately 8” clear distance to the subject structure. The connection wing to the 2-story Building 7 to the south of Building 3 is constructed with approximately 8” clear distance to the subject structure. Proximity to adjacent structure on both sides is less than the required 10.9” (4 percent of 22’-8” concourse height), and is non-compliant. The seismic gap between Building 6 and Building 3 is insufficient per the Tier 1 analysis as well.

There are no seismic joints between Building 3 and the four (4) steel framed appendages. The stiff concrete shear walls in plane with the flexible steel moment frames do not have compatible stiffness. Most of the seismic forces generated from the mass of the steel appendages will be transferred through ledger connections and braced by the stiffer concrete shear walls. Tier 2 analysis shows that the heavily reinforced concrete shear wall structure is adequate to resist the increase in seismic mass from the four (4) appendages, but the existing ledger connections between the structures are insufficient. The demand/capacity ratio (DCR) of the ledger anchorage at the roof is $2.35 > 1.0$ and at the floor is $3.25 > 1.0$.

RECOMMENDATION: Additional more detailed Tier 3 analysis must be performed to approximate horizontal movement of each structure during a seismic event including the strengthening effects of retrofit options provided in this report. The current scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3). Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause life safety structural concerns within the subject building. Egress issues are recommended to be further analyzed.

There are several alternate recommended solutions at the steel framed appendages around Building 3. The appendages may be seismically separated from the structure and roof/floor decking re-supported with new steel framing. Alternatively, the existing ledger connections at the roof and floor can be strengthened with new anchorage or the steel appendages can be demolished as noted below. See Appendix G for plans and details specifying retrofit options.

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Buildings 3, 4 and 5

The ASCE 41-13 Tier 1 Building Type Specific Checklist indicates the 1956 concrete Buildings 3, 4, and 5 as non-compliant in two (2) areas for Life Safety Performance.

- a. COMPLETE FRAMES (ASCE Section A.3.1.6.1) - *“Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.”* Numerous exterior concrete shear walls support gravity concrete beams. Tier 2 analysis performed (see calculations in Appendix F) to confirm existing concrete shear walls adequate to support combined gravity and seismic demands.

RECOMMENDATION: Per Tier 2 analysis no additional strengthening required.

- b. COUPLING BEAMS (ASCE Section A3.2.2.3) – *“The stirrups in coupling beams over means of egress are spaced at or less than $d/2$ and are anchored into the confined core of the beam with hooks of 135 degrees or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning.”* Stirrups in coupling beams over means of egress do not have 135 degree hooks, which may result in less ductile behavior and added damage/debris.

RECOMMENDATION: Jacket coupling beams with fiber reinforced polymer (FRP) strips or steel plates as means of controlling debris over means of egress per details and plan locations specified in Appendix G.

Building 6

The ASCE 41-13 Tier 1 Building Type Specific Checklist indicates Building 6, the 1961 concrete masonry addition to Building 5, as non-compliant/unknown in three (3) areas for Life Safety Performance.

- a. WALL ANCHORAGE (ASCE Section A.5.1.1) – *“Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7.”* Exterior CMU walls are adequately doweled into the roof slab per the Quick Check. However, the slab itself is connected to the existing building upon which it relies for out-of-plane support using archaic expansion anchors with no reliable tensile capacity. The steel ledger at this connection is visibly deteriorated by rust.

RECOMMENDATION: More extensive Tier 2 evaluation procedures were not performed as the capacity of the archaic, unreliable anchorage system would not have been justified. Retrofit the steel ledger connection by installing a new continuous steel angle below welded to the existing ledger and attached to the Building 5 concrete wall with adhesive anchors per schematic retrofit detail 1/SSK STEEL LEDGER RETROFIT.

- b. TRANSFER TO SHEAR WALLS (ASCE Section A.5.2.) – *“Diaphragms are connected for transfer of seismic forces to the shear walls.”* Exterior CMU walls are adequately doweled into the roof slab for shear transfer on the south side. However, on the north side, the slab is connected to the existing building upon which it relies for in-plane support using archaic expansion anchors with no reliable capacity. The steel ledger at this connection is visibly deteriorated by rust and the dry packed expansion anchors, while not observed due to obstructions, are likely compromised.

RECOMMENDATION: See (a) recommendations above.

- c. FOUNDATION DOWELS (ASCE Section A.5.3.5) – “*Wall reinforcement is doweled into the foundation.*” No dowels were specified in the existing drawings provided at the pilaster/boundary element of the CMU wall piers and the dowels were not observed in the field as no destructive testing was performed. Because the walls have a height-to-width ratio of approximately 3:1 their performance in a seismic event is heavily dependent on their flexural capacity in addition to shear. The flexural capacity of the CMU wall pier can only be developed if the boundary steel is adequately doweled into the foundation.

RECOMMENDATION: If the wall piers in their current condition are to be evaluated further per Tier 2, additional information on the as-built condition of the wall pier boundary steel dowels is required. Recommend selective demolition is performed to the base of at least two CMU wall piers to determine as-built dowel information then further analysis to determine adequacy.

If selective demolition is not performed, or dowels are determined to be inadequate for the Tier 2 seismic demands, we recommend two spandrel/window bays be demolished and concrete shear wall infill with adhesive dowels to the adjacent CMU wall piers and the stem wall below be installed per schematic retrofit detail 2/SSK CONCRETE SHEAR WALL INFILL RETROFIT.

Building 3 Steel Appendages

The ASCE 41-13 Tier 1 Building Type Specific Checklist indicates the four (4) 1987 steel structure appendages to Building 3 as non-compliant in six (6) areas for Life Safety Performance.

- a. DRIFT CHECK (ASCE Section A.3.1.3.1) – “*The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.5.3.1, is less than 0.025.*” If the appendages resisted their full seismic load, the frames typically do not comply with Tier 1 or Tier 2 drift evaluation in the longitudinal (East-West) direction. However, the appendages may not be fully seismically loaded since attached to and restrained by Building 3. The stiff concrete shear walls of Building 3 that are in plane with the flexible steel moment frames do not have compatible stiffness. The majority of seismic forces from the steel appendages will shift to the concrete shear walls requiring the ledger connection to transfer the load.

RECOMMENDATION: If the steel appendages remain connected to the concrete structure, add new stiffer lateral systems (steel plate shear wall/brace frame) parallel to the exterior concrete wall of Building 3 to more closely match its stiffness and reduce drift. In addition it is recommended that the ledger connection between the structures be strengthened. If the steel appendages are seismically separated from Building 3, strengthen the moment frame beams, columns, and connections in both directions to reduce drifts. Alternatively, the steel appendages can be demolished. See Appendix G for plans and details specifying retrofit options.

- b. FLEXURAL STRESS CHECK (ASCE Section A.3.1.3.3) – “*The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9, is less than F_y . Columns need not be checked if the strong column–weak beam checklist item is compliant.*” If the appendages were detached from Building 3, the majority of the beams and columns do not comply with the Tier 1 and Tier 2 evaluation. The average demand/capacity ration (DCR), using the average maximum member stress, is approximately 2.16 > 1.0.

RECOMMENDATION: If the steel appendages are seismically separated from Building 3, strengthen moment frame beams and columns with welded plates on flange and/or webs. Completing a full Tier 3 analysis of the structure may reduce the required retrofit. The current

scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3). See Appendix G for plans and details specifying retrofit options.

- c. **MOMENT-RESISTING CONNECTIONS** (ASCE Section A.3.1.3.4) – *“All moment connections are able to develop the strength of the adjoining members based on 110% of the expected yield stress of steel per AISC 341, Section A3.2.”* The majority of beam to column connections are non-compliant in both directions per the Tier 1 and Tier 2 evaluation.

RECOMMENDATION: If steel appendages are seismically separated from Building 3, retrofit the beam to column connections by adding cover plates to the top and bottom beam flanges at the column interface which will deliver the full capacity of the beam section to the column. Completing a full analysis of the structure through Tier 3 may reduce the required retrofit. The current scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3). Completing a full analysis of the structure may reduce the required retrofit. See Appendix G for plans and details specifying retrofit options.

- d. **PANEL ZONES** (ASCE Section A3.1.3.5) - *“All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column.”* Column panel zones do not have adequate shear capacity per Tier 1 and Tier 2 evaluations where beams frame into both sides of the columns. Average demand/capacity ratio (DCR) is approximately $1.24 > 1.0$.

RECOMMENDATION: If the steel appendages are seismically separated from Building 3, retrofit columns with steel plates welded to each side of the web to increase panel zone strength. Completing a full analysis of the structure through Tier 3 may reduce the required retrofit. The current scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3). Completing a full analysis of the structure may reduce the required retrofit. See Appendix G for plans and details specifying retrofit options.

- e. **STRONG COLUMN—WEAK BEAM** (ASCE Section A.3.1.3.7) – *“The percentage of strong column—weak beam joints in each story of each line of moment frames is greater than 50%”.* W8x35 columns are weaker at roof W12x26 beams or typically where beams frame into each side of column at the roof and second floor. Maximum demand/capacity ratio (DCR) is approximately $2.04 > 1.0$.

RECOMMENDATION: If the steel appendages are seismically separated from Building 3, retrofit the beams in both directions by adding a reduced beam section “dog bone” cut (RBS) in the beam flanges away from the column face, forcing the rotational hinge condition to occur in the beam as dictated by modern codes effectively making the columns stronger than the beams (Strong Column – Weak Beam Theory). It should be noted that adding RBS cuts in beams can increase building drifts by approximately 10% which will need to be accounted for in determining extent of other moment frame retrofit work. Completing a full analysis of the structure through Tier 3 may reduce the required retrofit. The current scope for Tier 1 / Tier 2 analysis is considered a generalization of a detailed building analysis (Tier 3). See Appendix G for plans and details specifying retrofit options.

BUILDING 7 (1936 Original Hospital Building)Evaluation Overview

This seismic evaluation report for the existing building located at 3325 Chanate Road in Santa Rosa, CA, is based on the following:

- The American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI 41-13) Standard for Seismic Evaluation and Retrofit of Existing Buildings - Tier 1 Life Safety level structural evaluation criteria.
- Three site visits for general review of structures performed on 11/07/14, 11/14/14, and 11/21/14. No destructive testing or removal of finishes was performed or included in scope.
- Review of the following original drawings and report:
 - Structural drawings by John I Easterly, Architect, dated 10/21/35.
 - “Seismic Hazard Investigation, Community Hospital of Sonoma County, Santa Rosa, California” by H.J. Degenkob & associates, Engineers, dated 08/04/78
- Existing material properties as indicated in Appendix D.
- Review of the following geotechnical reports and hazard maps:
 - Site geologic Hazard investigations and project specific geotechnical reports as indicated in the Geotechnical Summary.
 - Liquefaction Susceptibility and Surface Fault Rupture hazard maps from Association of Bay Area Governments (ABAG).
- Review of non-structural elements is not included.

Structural System and Materials Description*General*

Building 7 was designed in 1935 and constructed in 1936. Originally designed as an “I”-shaped building with smaller projections from the center, Building 7 was constructed without the northeast wing, southeast wing, and part of the east central projection, as shown in the original drawings. The building is adjacent to the 1956 additions to the north and the east (Buildings 3 and 8). The building is two stories with a full footprint basement and crawlspace, with a total interior floor area of 37,130 square feet. (See Appendix C – Photos 1 & 2, and Appendix B – Schematic Site Plan).

The building structural system is unusual for its era since it utilizes open web steel joists, which were first offered in 1932, and light gage metal studs, which were not typically available until the late 1920s and early 1930s. The building is also the oldest and most ornate building on the Chanate Hospital campus, and thus may have the greatest historic value.

Roof Framing

The structure is approximately 30’-6” tall above grade at the ridge line, with a typical roof slope of 5:12. The roof structure is 2½” formed concrete, minimally reinforced with welded wire fabric, over sloped 12”-deep steel open web joists at 32” on center. The joists are top chord bearing on stud bearing walls at the exterior perimeter, are

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supported by steel channel beams at the interior corridor stud walls, and are supported by ridge-aligned steel angle trusses at the wings. The corridor channel beams span to double angle posts over the stud walls and the wing trusses span to interior steel columns. The interior stud bearing walls stop 18 inches above the ceiling level. The open attic area is braced with steel diagonal angles in both directions over the corridor walls. The flat ceilings are plaster over suspended metal lath.

First and Second Floor Framing

The floor structures are 2½" formed concrete, minimally reinforced with welded wire fabric, over 12"-deep steel open web joists at 32" on center, with story heights of 11'-4". The joists are top chord bearing on continuous steel angle ledgers that are welded to the sides of full height stud bearing walls. The concrete floor diaphragms are discontinuous at the interior metal stud bearing walls. The ceilings are plaster over metal lath. The basement below the elevated first floor is 7 feet tall, used for mechanical and storage space below the corridor and east side, and a 4 foot tall unfinished crawl space west of the corridor.

Walls

Typical exterior and interior bearing walls are light gage metal studs at 16" on center, with stucco or plaster over metal lath finish at the exterior or interior. Typical stud profile is 16gage thick by 6" deep, with 1" wide flanges (Appendix C – photo 9). The stud webs have large 3" maximum width triangular punch-outs and deformation of the remaining material, forming a truss-type configuration. Light "I"-columns occur at approximately 12' on center along the corridor stud walls, with a typical profile of 10gage thick by 6" deep, with 2" wide flanges and large triangular punch-outs. All metal stud walls run full height of the building, from 18 inches below the first floor to 18 inches above the second story ceiling. All full height stud walls disrupt the concrete diaphragms at the first and second floor levels. Partition stud walls not functioning as bearing walls do not run full height or interrupt the diaphragms.

Lateral Force Resisting System

The primary lateral force resisting system consists of braced wall panels with diagonal rod pairs that run through the punch-outs at the centerline of the studs. Rod sizes vary from 1/2" to 7/8", and braced wall panels vary from 8' to 14' long. Typical rod connections to the foundation are by means of lapping approximately 4 inches and welding to a rod stub that is cast vertically into the concrete basement wall and bent over to the brace angle (Appendix C – photo 10). Typical rod connections at the roof level are by means of running through and bolting to a transverse angle that bears against the side of stud, located eccentrically, 4 inches down from the top track (Appendix C – photos 5 & 6). Each braced wall panel has a single typical stud at the end for overturning forces, which is minimally welded to the sill track at the basement walls. Anchorage of the sill track consists of a single 1/2" diameter anchor bolt, located eccentrically, 2 inches away from the wall panel edge stud. Typical chords and collectors consist of continuous stud wall tracks at the open attic and continuous angle ledgers at the floors (Appendix C – photos 3, 4, & 9). Each type is spliced minimally, and the floor ledgers are interrupted at changes in framing direction and at re-entrant corners.

An additional lateral system, although un-designed and not constructed for lateral resistance, consists of the plaster and stucco which acts as shear material over the bearing stud walls. The steel lath was not observed to be systematically connected to the steel studs with closely spaced fasteners as would be required to develop shear capacity. Generally, stucco and plaster shear walls are an archaic lateral force resisting system that is brittle, does not perform well under cyclic loading, and is typically not used for design to modern building codes. However, plaster and stucco shear wall systems can possibly be strengthened by applying fiber reinforced polymer (FRP) with polymer fasteners to the wall studs, and such strengthening could potentially be considered as an alternative to the strengthening recommendations in this report.

Foundations

Typical foundations are 8 inch thick concrete basement walls that vary in height from 4 to 7 feet. Basement walls have continuous concrete spread footings that vary in width from 12 to 16 inches, and are embedded 12 inches

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below basement slab and several feet below grade at the exterior perimeter. Columns at the west wings bear on 15" square concrete pilasters with isolated pad footings.

Field Verification and Condition Assessment

The building appears to have generally been built per the original construction documents with the main exception of elimination of the northeast and southeast wings and part of the central projection as mentioned above. The lateral force resisting system is not well documented in the drawings, with "Typical Wall" elevations that schematically indicate the diagonal rods. Rod connections are not detailed and the number and location of braced wall panels is not shown on the plans. Field verification of the diagonal rods resulted in estimating 37 rod pairs in the north-south direction and 26 rod pairs in the east-west direction. Roughly 20 percent of the stud walls at the basement were obscured from view, requiring estimation of the total number of pairs: 31 of the estimated 37 north-south pairs were verified and 21 of the 26 east-west pairs were verified. Rod sizes at the longer panels are typically 7/8" diameter at the first story and 5/8" diameter at the second story. Shorter panels are typically 1/2" or 5/8" diameter in the north-south direction. All rod stubs at the basement walls are 7/8" diameter.

The braced wall panels and lateral systems appear to have been changed during construction from the design intent and improvised at many locations. The drawings call for 7/8" rods at the first story, yet three sizes were used. The drawings call for L2x2½ angle ledgers at each diaphragm level, yet L1x1 was used at the floors, and the angle was eliminated at the roof. Several locations, where braced wall panels were expected to be found, had rod stubs that were cast into the basement walls, but not used (Appendix C – photo 11). Several locations were found with rod ends that were bent and minimally welded to floor beams. End connection lap lengths and bearing angle size and configuration was found to be varied (Appendix C – photos 5, 6, 7, & 8). Many rods were found to be somewhat loose and movable by hand, and several locations were found with end nuts missing (Appendix C – photo 7). Loose rods allow a building to deflect before engagement of the lateral system, resulting in larger story drifts and increased damage during seismic events.

The building is generally in fair condition. The basement walls are in good condition with no signs of reinforcement corrosion and minimal cracking. However, several locations were found where door openings were saw-cut, presumably during mechanical renovations, without header reinforcement. The floor joists are in good condition with minor signs of corrosion, but several locations were found where chords were cut or bent to allow for installation of piping, presumably during renovations. The metal studs are in fair condition, with corrosion primarily at the bottom few inches at the basement walls, and with many locations found where the studs have been bent or cut. The metal stud sill tracks that attach the stud walls to the concrete basement walls are in poor condition, with widespread corrosion throughout the building (Appendix C – photos 10, 11, & 12). The metal stud top tracks in the attic space are in fair condition, with many locations found where bent or drilled for large pipes (Appendix C – photo 4). The floor slabs are in fair condition, with many locations found with spalls and with corroded reinforcement at the bottom of slab.

Although access was provided for field verification of conditions at the basement and at the attic space, second floor access was not possible due to an inability to perform destructive testing. For the purposes of this report, the second floor is assumed to be framed similarly to the first floor with rod connections similar to those at the roof.

Material Properties

Basic properties for existing structural materials through ASCE 41 code prescribed minimum values utilized in the analysis calculations can be found in Appendix D.

Building Type

Per ASCE/SEI 41-13, this building does not conform to a standard classification type. Diagonal rod wall panels are typically not used for design to modern building codes, have not historically been used for lateral force resisting systems for buildings, and have not been addressed by ASCE/SEI 41-13 because of their rarity.

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However, for purposes of Tier 1 analysis for estimation of adequacy of the lateral force resisting system, the building most closely resembles **Building Type S2: Steel Braced Frames with Stiff Diaphragms**. (See Appendix E). As described by ASCE/SEI 41-13: *These buildings have a frame of steel columns, beams, and braces. Braced frames develop resistance to seismic forces by the bracing action of the diagonal members. The braces induce forces in the associated beams and columns such that all elements work together in a manner similar to a truss with all element stresses being primarily axial. Diaphragms transfer seismic loads to braced frames. The diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames. The foundation system may consist of a variety of elements... Configuration and design of braced frames... are:*

- *Concentrically Braced Frames: Component worklines intersect at a single point or at multiple points such that the distance between intersecting worklines (or eccentricity) is less than or equal to the width of the smallest component connected at the joint.*

Historical Performance

This building does not conform to a standard classification type and, therefore does not have a well-known performance track record.

Benchmark Buildings

In addition to classifying buildings by type of construction, ASCE 41 identifies 'Benchmark Buildings' for each type. The detailing of seismic force resisting systems in Benchmark Buildings is generally considered to meet the performance requirements of ASCE 41. When a building is determined to meet Benchmark Building requirements through field verification of construction compliant with benchmark code requirements, only review of foundation and non-structural elements is required. Even though a building appears to meet the benchmark criteria, a full analysis may still be recommended under certain circumstances.

For building type S2, the 1997 UBC seismic design provisions are referenced as the oldest permitted standard. Since the subject building was constructed in 1936 and does not conform to a classification type S2, it does not meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

Findings and Recommendations

The ASCE 41-13 Tier 1 Basic Life Safety and Building Type Specific Checklists indicate the primary building structure as non-compliant in twelve (12) areas for Life Safety Performance. Note that eight of the areas are derived from the S2 Building Type Checklist, which is not directly applicable to this building type. However, engineering judgment determines that these items are representative of criteria that should be met for the building's specific lateral force resisting system and are therefore worth examining.

- a. **LOAD PATH** (ASCE Section A.2.1.1) – *"The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation."* The lateral force resisting system, consisting of diagonal rod braced wall panels, has a lack of load path. Chords and collectors are neither well-defined nor detailed for seismic forces at the roof diaphragms. Minimal structural continuity exists, consisting of thin gage metal tracks with long unbraced lateral lengths and minimal splices (Appendix C – photos 3 & 4). Chords and collectors are neither well-defined nor detailed for seismic forces at the floor diaphragms. Minimal partial structural continuity exists, consisting of small steel ledger angles that are discontinuous at re-entrant corners and changes in framing direction (Appendix C – photo 9). Floor diaphragms are discontinuous at interior stud walls, which run full height of the building, and do not have a shear transfer load path across the interior diaphragm gaps. Shear transfer from the roof diaphragm to the braced wall panels is neither well-defined nor detailed for seismic forces. Minimal lateral connection consists of weak-axis bending of thin gage metal roof joist supports (Appendix C –

photo 3). Shear transfer from the second floor diaphragm to the braced wall panels is neither well-defined nor detailed for seismic forces. Minimal lateral connection consists of weak axis bending of thin gage metal stud webs. Shear transfer from the first floor diaphragms to the foundation stem walls is neither well-defined nor detailed for seismic forces. Minimal lateral connection consists of weak axis bending of thin gage metal studs (Appendix C – photo 9). Development of the braced wall panel rod ends is neither well-defined nor detailed for seismic forces. Minimal connections rely upon eccentric force concentrations applied to the face of thin gage metal stud webs (Appendix C – photos 5, 6, & 7). Transfer of the wall panel overturning forces from edge studs is neither well-defined nor detailed for seismic forces. Minimal connection consists of eccentric loading and weak axis bending of thin gage metal tracks.

RECOMMENDATION: Even if all load path issues are resolved by means of repair, the diagonal rods and wall panel edge studs are substantially deficient as noted in (d) and (e) below. Considering the long list of systemic deficiencies including load path as noted above and areas noted in (c), (d), (e), (f), (g), (h), (i), (j), and (k) below, the building does not contain a substantial lateral force resisting system. Therefore, whole systems recommendations must be considered. One of the following options is recommended:

- 1) Demolish the building.
 - 2) Provide an entirely new lateral force resisting system within the building, consisting of structural steel braced frames, structural steel chords and collectors, and metal stud blocking at all diaphragm discontinuities. The construction impacts for this retrofit are intensive. See Appendix G – Strengthening Sketches.
 - 3) Selectively demolish portions of the building and retrofit the remaining areas that are chosen to be kept for program functionality or historical value purposes.
 - 4) Abandon the building and provide adequate barrier to limit access or proximity to the building on all sides. Adjacent buildings 3 and 8 should not rely upon Building 7 for egress nor should egress be allowed within proximity of Building 7 due to potential collapse. Adjacent buildings and covered walkways should also be strengthened as required to withstand impact due to potential collapse of Building 7.
- b. ADJACENT BUILDINGS (ASCE Section A.2.1.2) – *“The clear distance between the building being evaluated and any adjacent building shall be greater than 4 percent of the height of the shorter building.”* The 8” clear gap at adjacent Building 3 does not meet the minimum Tier 1 required clear separation for independent seismic performance. The 8” gap is less than 4% of 24 feet (11.5 inches). The 12” clear gap at adjacent Building 8 does meet the minimum Tier 1 requirement. However, by observation, the clear separation at all locations will be insufficient to protect Buildings 3 and 8 and the covered walkways from damage due to potential collapse of Building 7 if abandoned as optionally recommended in (a) above.

RECOMMENDATION: If a new lateral force resisting system is provided as optionally recommended in (a) above, then the clear separations are recommended to be analyzed for adequacy. However, remediation of this deficiency may not be feasible.

- c. TORSION (ASCE Section A.2.2.7) – *“The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.”* As noted in (a) above, the floor diaphragms are discontinuous at interior stud walls. Therefore, each floor consists of roughly 15 independent diaphragms that are not interconnected for shear. Many of the diaphragms are eccentrically braced by observation.

RECOMMENDATION: See (a) recommendations above.

- d. COLUMN AXIAL STRESS CHECK (ASCE Section A.3.1.3.2) – *“The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$.”* Overturning axial forces are taken by a single typical stud at the braced wall panel edges. At a Tier 1 analysis force level, the demand-to-capacity ratio varies from 8.4 to 9.5 when compared to the $0.30F_y$ criteria (up to $2.85F_y$), which is substantially deficient (See Appendix F). The wall panel edge studs require replacement with structural steel.

RECOMMENDATION: See (a) recommendations above.

- e. BRACE AXIAL STRESS CHECK (ASCE Section A.3.3.1.2) – *“The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.5.3.4, is less than $0.50F_y$.”* At a Tier 1 analysis force level, the demand-to-capacity ratio of the diagonal rods varies from 7.5 to 10.5 when compared to the $0.50F_y$ criteria (up to $5.25F_y$), which is substantially deficient (See Appendix F). The diagonal rods require replacement with structural steel.

RECOMMENDATION: See (a) recommendations above.

- f. TRANSFER TO STEEL FRAMES (ASCE Section A.5.2.2) – *“Diaphragms are connected for transfer of seismic forces to the steel frames.”* As noted in (a) above, the lack of direct, well defined load path from diaphragms to collectors results in a lack of seismic force transfer to the wall panels.

RECOMMENDATION: See (a) recommendations above.

- g. STEEL COLUMNS (ASCE Section A.5.3.1) – *“The columns in seismic-force-resisting frames are anchored to the building foundation.”* As noted in (a) above, the lack of direct, well defined load path from edge stud to eccentrically placed anchor bolt results in a lack of anchorage.

RECOMMENDATION: See (a) recommendations above.

- h. COMPACT MEMBERS (ASCE Section A.3.3.1.7) – *“All brace elements meet compact section requirements set forth by AISC 360, Table B4.1.”* The single typical stud at the braced wall panel edges are thin gage and are non-compact by observation.

RECOMMENDATION: See (a) recommendations above.

- i. CONNECTION STRENGTH (ASCE Section A.3.3.1.5) – *“All the brace connections develop the yield capacity of the diagonals.”* Development of the braced wall panel rod ends relies upon eccentric force concentrations applied to the face of thin gage metal stud webs. By observation, connections do not develop the rods.

RECOMMENDATION: See (a) recommendations above.

- j. COMPACT MEMBERS (ASCE Section A.3.3.1.7) – *“All brace elements meet section requirements set forth by AISC 341, Table D1.1, for moderately ductile members.”* The single typical stud at the braced wall panel edges are thin gage and non-ductile by observation.

RECOMMENDATION: See (a) recommendations above.

- k. CONCENTRICALLY BRACED FRAME JOINTS (ASCE Section A.3.3.2.4) – “*All the diagonal braces shall frame into the beam–column joints concentrically.*” As noted in (a) above, the diagonal rods are eccentrically connected at their ends.

RECOMMENDATION: See (a) recommendations above.

- c. SURFACE FAULT RUPTURE (ASCE Section A.6.1.3) – “*Surface fault rupture and surface displacement at the building site are not anticipated.*” The site is located in the Alquist-Priolo special study zone per the California Division of Mines and Geology (CDMG) Santa Rosa Quadrangle Map published in 1983. Multiple geologic hazard evaluations and geotechnical reports completed by five geotechnical firms were reviewed for this site. The oldest reviewed report was completed in 1978 while the most recent was completed in 2002. The exact location of faults in the area of the Chanate Hospital buildings is not clearly determined by the reports due to differing data and conclusions. The 2002 report was performed by Rutherford & Chekene (R&C) and included a geologic hazard evaluation by Gilpin Geosciences. Gilpin Geosciences Report summarizes the surface fault rupture hazard:

“Based on the preponderance of lineaments and other fault-related features observed by Gilpin Geosciences and others in the site vicinity, along with the lack of clear resolution of differing interpretations of onsite and offsite geologic structures, we conservatively judge the overall potential for fault rupture at the site to be high. There may exist areas within the site that are sufficiently free of active faults so as to allow future construction of structures for human occupancy.”

A lineament is a feature in a landscape which is an expression of an underlying geologic structure such as a fault. The 1978 Cooper Clark & Associates report estimated the maximum potential offset at the surface as 25 inches horizontally and 2.5 inches vertically. The report summarized the potential surface rupture behavior:

“If surface rupture were to occur along the part of the fault near the hospital, the displacement could occur along the relatively well located trace mapped to the west, along the approximately located trace mapped near this site, or along other traces, such as those found nearby in our trenches – a zone probably 980 feet wide.”

Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred. Four of the five geotechnical engineers that provided information for the site concluded there were likely fault traces or fault related features extending through at least some portion of the site. Rutherford & Chekene’s (R&C) conclusions for the site varied. In the 1986 report R&C concluded there were fault traces on the site but they are not considered active. Reports completed between 1987 and 1992 concluded there were no fault traces at the specific reviewed locations and the entire health care facility was suitable for development from a geologic/geotechnical viewpoint. The final R&C report in 2002 concluded it was likely that fault traces projected through the site and under the existing Medical Center complex. Additionally the report stated there was a high potential for surface rupture. There does appear to be areas where fault traces may be less likely to occur based on the fault maps provided. See the geotechnical summary for a complete discussion of all reviewed information pertaining to surface fault rupture.

The 1978 Cooper Clark & Associates report was the most extensive study of potential fault traces on the site that was reviewed. R&C performed many investigations with differing conclusions however the latest R&C report reviewed stated the overall potential for fault rupture at the site to be high. This finding is in line with Cooper Clark, Herzog and HLA (Herzog and HLA reports are for buildings not included in the scope of work for this project). The following is a summary of

potential fault traces as it affects the building. See the Geotechnical Summary for a complete summary of all reviewed reports.

If the fault trace projections from the 1978 Cooper Clark & Associates report were extended south they would likely pass through the North and South wings of the 1936 building. The 1986 R&C report found several fault traces that approximately aligned in location and orientation with the Cooper Clark report that would project beneath this building; however they classified the faults as non-active (more than 11,000 years old). The 2002 R&C report stated it is prudent to assume that active faults may project under the existing Medical Center complex. The 2002 Rutherford and Chekene / Gilpin Geosciences report found three offsets in the sidewalk along the south side of the building and noted they may be evidence of fault creep. During site review for this project two of these offsets were located which approximately align with the projected fault traces. This may be evidence of fault creep on the site.

RECOMMENDATION: Based on the information contained in the reviewed reports and the CDMG maps, the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. Based on the extreme deficiencies of the building system as discussed in the Structural Findings and Recommendations above, fault offsets are likely to exacerbate the already large collapse potential. If a new lateral force resisting system is provided as optionally recommended, significant fault rupture within the building envelope could still likely damage the building beyond repair or future use. Thus, a comprehensive review would be prudent in the case of Building 7 to determine if strengthening is warranted.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

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BUILDING 8 (1956 Kitchen/Storage Building)

Evaluation Overview

- The American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI 41-13) Standard for Seismic Evaluation and Retrofit of Existing Buildings - Tier 1 and Tier 2 (non-compliant items only), Life Safety level structural evaluation criteria.
- A site visit for general review of structures performed on 11/7/14. No destructive testing or removal of finishes was performed or included in scope.
- Review of following original drawings:
 - Structural drawings by Art B. Smith Structural Engineers (1953). Complete Architectural floor plans and elevations were not available for this building.
- Existing material properties from the original construction drawings and ASCE 41 default values as indicated in Appendix D.
- Review of following geotechnical reports and hazard maps:
 - Site geologic Hazard investigations and project specific geotechnical reports as indicated in the Geotechnical Summary.
 - Liquefaction Susceptibility and Surface Fault Rupture hazard maps from Association of Bay Area Governments (ABAG).
- Review of non-structural elements is not included.

Structural System and Materials Description

General

The Kitchen Building (Building 8) was designed in 1953 and built in 1956. The structure is a long rectangular building with approximately 8600 square feet.

Roof Framing

The structure is approximately twelve feet tall with top of flat roof occurring at approximately eleven feet above first floor level. The roof structure is wood framed consisting of 1x diagonal sheathing over 2x14 joist at 16 inches on center supported by exterior concrete walls and a central steel beam. The central steel beams are supported by steel pipe columns at eighteen feet on center.

Walls

Typical exterior concrete walls are eight inches thick reinforced cast-in-place concrete walls with cast-in-place twelve inch columns at eighteen feet on center. At mid-length of the structure there is an interior eight inch thick cast-in-place concrete wall, the remaining interior walls are wood framed partitions. The typical concrete walls are reinforced with #4 bars at ten inches on center each way. The reinforcing at concrete columns varies but at a minimum is four #7 vertical bars with #3 ties at eight inches on center.

Lateral Force Resisting System

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The primary lateral force resisting system for the structure is reinforced concrete shear walls. Roof diaphragm is 1x6 diagonal sheathing.

Foundations

Foundations are continuous fourteen wide spread concrete footings at concrete walls with a wire mesh reinforced four inch thick concrete slab on grade floor system. Interior pad footings occur at steel pipe columns.

Field Verification and Condition Assessment

The structure appears in generally good structural condition with minimal structural damage or deterioration apparent. The building appears to be constructed in general accordance with the provided structural drawings. At the east side of the structure a portion of the covered loading dock area was infilled with wood framed walls that were not part of the original construction per the documents reviewed.

Material Properties

Basic properties for existing structural materials found on existing building documentation, through testing or ASCE 41 code prescribed minimum structural values utilized in the analysis calculations can be found in Appendix D.

Building Type

Per ASCE/SEI 41-13, this building can be classified as **Building Type C2A: Concrete Shear Walls With Flexible Diaphragms**. As described by ASCE/SEI 41-13: *‘These buildings have floor and roof framing that consists of wood sheathing, or cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs that have large aspect ratios, and are flexible relative to the walls. Buildings may also have steel beams, columns, and concrete slabs for the gravity framing. Floors are supported on concrete columns or bearing walls. Seismic forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced but often extend throughout the building. In more recent construction, shear walls occur in isolated locations, are more heavily reinforced and have concrete slabs which are stiff relative to the walls. The foundation system may consist of a variety of elements.’*

Historical Performance

While cast-in-place concrete shear wall systems with wood diaphragms have traditionally proved adequate for gravity loading, older buildings have not performed well during an earthquake. Shear wall elements perform relatively well in-plane for earthquake events provided adequate shear wall length is maintained without localized stresses in short wall piers, and provided there are no significant plan or vertical discontinuities such as a difference in stiffness between shear walls of adjacent levels in a multi-story structure. Positive wall-to-diaphragm connections are critical to performance and often heavy concrete walls are not adequately anchored to light wood-framed roof diaphragms.

Building collapse is rarely attributed to a failure of the wood framed diaphragms or concrete walls, but rather to failure in wall-to-diaphragm elements in the load path, such as collectors or connections between diaphragms and vertical elements, and out of plane wall anchorage. The most common failure type is an outward collapse of the exterior concrete walls caused by separation of the walls from the floor and roof diaphragms. In light of this typical failure method, current iterations of the building code require more stringent detailing requirements for the roof to wall connection. Even in highly redundant buildings with typically long concrete wall lengths and low shear stresses, some level of structural retrofit is usually required to ensure adequate building performance in a seismic event. The addition of interior shear walls is also a viable retrofit technique for low capacity diaphragms.

Benchmark Buildings

In addition to classifying buildings by type of construction, ASCE 41 identifies ‘Benchmark Buildings’ for each type. The detailing of seismic force resisting systems in Benchmark Buildings is generally considered to meet the

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performance requirements of ASCE 41. When a building is determined to meet Benchmark Building requirements through field verification of construction compliant with benchmark code requirements, only review of foundation and non-structural elements is required. Even though a building appears to meet the benchmark criteria, a full analysis may still be recommended under certain circumstances.

For building type C2A, the 1985 NEHRP seismic design provisions are referenced as the oldest permitted standard. Since, the subject building was constructed in 1956, and per the provided documentation was constructed under the 1952 code, it does not meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

Findings and Recommendations

The ASCE 41-13 Tier 1 Basic Life Safety and Building Type Specific Checklists indicate the primary building structure as non-compliant in four (4) areas for Life Safety Performance.

- a. ADJACENT BUILDINGS (ASCE Section 4.3.1.2) – *“The clear distance between the building being evaluated and any adjacent building shall be greater than 4 percent of the height of the shorter building for Life Safety and Immediate Occupancy.”* The covered walkway that is part of the Kitchen building is framed without a sufficient separation to the adjacent 1936 building. Proximity to adjacent structure is less than the required 4 percent of walkway height, and is non-compliant.

RECOMMENDATION: Additional analysis may be performed to estimate horizontal movement in a seismic event. Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause life safety structural concerns within the subject building. Egress issues are recommended to be further analyzed.

- b. WALL ANCHORAGE (ASCE Section 4.6.1.1) – *“Exterior concrete or masonry walls, that are dependent on the diaphragm for lateral support, shall be anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed in to the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check Procedure of Section 3.5.3.7.”* Existing wall anchorage of longitudinal concrete walls to the roof framing does not meet the strength requirements of the Quick Check procedure. The capacity of the existing nails are deficient, however the remaining connection elements are adequate per the quick check procedure.

Tier 2 analysis of existing out of plane wall anchorage of the longitudinal walls to the roof indicate they are not adequate for the loading required to meet Life Safety standards, as noted above.

RECOMMENDATION: Increase the connection capacity of the existing wall anchors or add anchors. (See Schematic Repair Details Appendix G).

- c. DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. The roof diaphragm is diagonal sheathing which spans greater than 40 feet in each direction with a worst case span of 110 feet. Aspect ratios are less than 4-to-1 and are compliant. The diaphragms are not adequate per the Tier 2 analysis.

RECOMMENDATION: Provide a new plywood diaphragm over the existing diagonal sheathing for the entire building. Roofing replacement is required. (See Schematic Repair Details Appendix G).

- d. SURFACE FAULT RUPTURE (ASCE Section A.6.1.3) – “*Surface fault rupture and surface displacement at the building site are not anticipated.*” The site is located in the Alquist-Priolo special study zone per the California Division of Mines and Geology (CDMG) Santa Rosa Quadrangle Map published in 1983. Multiple geologic hazard evaluations and geotechnical reports completed by five geotechnical firms were reviewed for this site. The oldest reviewed report was completed in 1978 while the most recent was completed in 2002. The exact location of faults in the area of the Chanate Hospital buildings is not clearly determined by the reports due to differing data and conclusions. The 2002 report was performed by Rutherford & Chekene (R&C) and included a geologic hazard evaluation by Gilpin Geosciences. Gilpin Geosciences Report summarizes the surface fault rupture hazard:

“Based on the preponderance of lineaments and other fault-related features observed by Gilpin Geosciences and others in the site vicinity, along with the lack of clear resolution of differing interpretations of onsite and offsite geologic structures, we conservatively judge the overall potential for fault rupture at the site to be high. There may exist areas within the site that are sufficiently free of active faults so as to allow future construction of structures for human occupancy.”

A lineament is a feature in a landscape which is an expression of an underlying geologic structure such as a fault. The 1978 Cooper Clark & Associates report estimated the maximum potential offset at the surface as 25 inches horizontally and 2.5 inches vertically. The report summarized the potential surface rupture behavior:

“If surface rupture were to occur along the part of the fault near the hospital, the displacement could occur along the relatively well located trace mapped to the west, along the approximately located trace mapped near this site, or along other traces, such as those found nearby in our trenches – a zone probably 980 feet wide.”

Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred. Four of the five geotechnical engineers that provided information for the site concluded there were likely fault traces or fault related features extending through at least some portion of the site. Rutherford & Chekene’s (R&C) conclusions for the site varied. In the 1986 report R&C concluded there were fault traces on the site but they are not considered active. Reports completed between 1987 and 1992 concluded there were no fault traces at the specific reviewed locations and the entire health care facility was suitable for development from a geologic/geotechnical viewpoint. The final R&C report in 2002 concluded it was likely that fault traces projected through the site and under the existing Medical Center complex. Additionally the report stated there was a high potential for surface rupture. There does appear to be areas where fault traces may be less likely to occur based on the fault maps provided. See the geotechnical summary for a complete discussion of all reviewed information pertaining to surface fault rupture.

The 1978 Cooper Clark & Associates report was the most extensive study of potential fault traces on the site that was reviewed. R&C performed many investigations with differing conclusions however the latest R&C report reviewed stated the overall potential for fault rupture at the site to be high. This finding is in line with Cooper Clark, Herzog and HLA (Herzog and HLA reports are for buildings not included in the scope of work for this project). The following is a summary of potential fault traces as it affects the building. See the Geotechnical Summary for a complete summary of all reviewed reports.

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This building (Kitchen and Storage Bldg 8) appears to be located to the east of all fault traces found on the site based on the reviewed reports. The building is the only building reviewed which is entirely in the area “less likely” for potential fault rupture

RECOMMENDATION: Based on the information contained in the reviewed reports and the CDMG maps, the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly define and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Relocation of the building is not likely a feasible solution, should fault rupture be determined conclusively. Remediation of this deficiency is likely not feasible. Based on size and orientation of the building, and redundancy of the systems minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing and non-structural (MEP and architectural framing); however, the structure overall is not likely to collapse. Significant fault rupture within the building envelope is likely to damage the building beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

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BUILDING 9 (1987 Ambulance Canopy)

Evaluation Overview

This seismic evaluation report for the existing building located at 3325 Chanate Road in Santa Rosa, CA, is based on the following:

- The American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI 41-13) Standard for Seismic Evaluation and Retrofit of Existing Buildings - Tier 1 and Tier 2 (non-compliant items only), Life Safety level structural evaluation criteria.
- A site visit for general review of structures performed on 11/7/14. No destructive testing or removal of finishes was performed or included in scope.
- Review of following original drawings:
 - Structural drawings by MKM & Associates: Civil and Structural Engineers (1987).
 - Architectural drawings by Lawry Coker DeSilva Architects A.I.A. (1987)
- Existing material properties as indicated in Appendix D.
- Review of following geotechnical reports and hazard maps:
 - Site geologic Hazard investigations and project specific geotechnical reports as indicated in the Geotechnical Summary.
 - Liquefaction Susceptibility and Surface Fault Rupture hazard maps from Association of Bay Area Governments (ABAG).
- Review of non-structural elements is not included.

Structural System and Materials Description

General

The Ambulance Canopy/Dock (Building 9) was designed in 1987. The canopy is a one story steel structure with an elevated concrete loading dock previously used for ambulance offload to the 1956 Emergency Room and 1987 Observation Room/Restroom additions (see Photos 1 & 2 – Appendix C). The canopy is irregularly shaped and approximately 625 square feet.

A larger ambulance canopy built in approximately the 1970's in the same location was reviewed in a 1978 H. J. Degenkolb & Associates, Engineers seismic hazard investigation report for the county and was deemed a probable collapse hazard in a seismic event due to direct connections to three seismically isolated structures. The direct connections would tear the canopy apart as the attached structures would move independently. This canopy was removed prior to the construction of the current canopy structure designed by MKM Structural Engineers. Removal of the condemned canopy was verified in field as the removed connections were still visible on the 1956 Emergency Room Addition concrete masonry unit wall (see Photos 7 & 8 – Appendix C).

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Roof Framing

The structure is approximately 13'-0" tall with top of flat roof occurring at approximately 11'-0" above first floor dock level. The roof structure is steel framed consisting of Verco Type N 20 GA metal deck spanning 13 feet maximum over TS8x6x3/16 tube steel supported by four TS6x6x3/16 tube steel columns. The tube steel beams are connected to the columns with four stiffened steel angle seats fillet welded to the columns and beams (see **Photo 3**). Direct welds from the beams to the columns were detailed in the MKM 1987 structural drawings but were not observed due to visual obstruction by the angle seats.

Loading Dock Construction

The elevated ambulance loading dock is approximately 22 inches tall. The dock is constructed of a 6 inch slab with #4 reinforcing bars at 18 inches on center each way. The slab is supported on fill retained by 6 inch stem walls with #4 reinforcing bars at 12 inches on center each way. Vertical stem wall reinforcing is terminated in the 6 inch loading dock slab above with 18 inch 90 degree hooks.

Lateral Force Resisting System

The primary lateral force resisting system for the canopy is the four TS6x6x3/16 cantilevered columns. The columns have a point of resistance above the base attachment at the corners of the loading dock slab with two 3 inch by 9 inch $\frac{1}{2}$ " plates welded to two $\frac{1}{2}$ " diameter studs embedded 3 inches into the slab. The columns extend below the loading dock into 18 inch square reinforced pedestals that extend 6 inches above the adjacent pavement (see Photo 4 – Appendix C). The columns are attached at the foundations below with a $\frac{1}{2}$ " base plate and four $\frac{3}{4}$ " \varnothing by 12 inch long anchor bolts. The attachment to the slab in combination with the anchor bolts to the foundation below create a propped cantilever action providing the "fixed" base moment resistance required for cantilevered column systems.

Foundations

The loading dock stem walls are supported on 12 inch wide by 12 inch deep continuous footings. The TS6x6x3/16 columns are supported on 24 inch square pad footings with three #4 reinforcing bars each way.

Field Verification and Condition Assessment

The structure appears in generally good structural condition with minimal structural damage or deterioration apparent, and appears to be constructed in general accordance with the provided structural drawings. The beams are attached to the columns with steel angle seats as noted above that were not included in the original construction documents. Column attachment plates to the loading dock slab were not observed during the review due to obstructions and are recommended to be verified in field by pachometer testing/scanning if needed for more extensive evaluation.

Material Properties

Basic properties for existing structural materials found on existing building documentation, through testing or ASCE 41 code prescribed minimum structural values utilized in the analysis calculations can be found in Appendix D.

Building Type

In order to perform the ASCE 41-13 analysis the ambulance canopy needed to be assigned a building classification. As cantilevered columns is not an ASCE prescribed building type, engineering judgment and past experience was used to classify the canopy as most similar to ASCE 41-13 **Building Type S1A: Steel Moment Frames with Flexible Diaphragms** where the moment resisting connection is the base connection of the cantilevered columns.

As described by ASCE/SEI 41-13: *'These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of un-topped metal deck (or with lightweight insulating concrete fill)*

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supported on steel beams, open web joists, or steel trusses. Seismic forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of wood framing; un-topped metal deck; or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping and are flexible relative to the frames. Where the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. The foundation system may consist of a variety of elements.'

Historical Performance

Due to the poor past performance of cantilevered column systems current building design codes significantly increase their relative design seismic loads. However, this structure is small, one-story and lightweight: therefore is not subject to large seismic forces. Cantilevered columns are a flexible system and subject to large story drifts. Cantilevered columns are designed to remain elastic through expected seismic demands as ductile yielding of the column base would lead to instability of the structure and probable loss of vertical load carrying capacity. The critical connection of these systems is the base connection of the column. The amount of fixity provided by the base connection is a large contributor to the drift of the structure under seismic demands as small rotations of the column base can project up to large translational displacements of the roof level. The base connection must also be designed to resist shear and moment forces associated with developing the bending demands on the column or the structure would be susceptible to premature loss of lateral load carrying capacity.

Benchmark Buildings

In addition to classifying buildings by type of construction, ASCE 41 identifies 'Benchmark Buildings' for each type. The detailing of seismic force resisting systems in Benchmark Buildings is generally considered to meet the performance requirements of ASCE 41. When a building is determined to meet Benchmark Building requirements through field verification of construction compliant with benchmark code requirements, only review of foundation and non-structural elements is required. Even though a building appears to meet the benchmark criteria, a full analysis may still be recommended under certain circumstances.

For building type S1A, the 1994 UBC seismic design provisions are referenced as the oldest permitted standard. Since, the subject building was constructed in 1987, and per the provided documentation was constructed under the 1979 UBC, it does not meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

Findings and Recommendations

The ASCE 41-13 Tier 1 Basic Life Safety and Building Type Specific Checklists indicate the primary building structure as non-compliant in three (3) areas for Life Safety Performance.

- a. ADJACENT BUILDINGS (ASCE Section 4.3.1.2) – *"The clear distance between the building being evaluated and any adjacent building shall be greater than 4 percent of the height of the shorter building for Life Safety and Immediate Occupancy."* The canopy is constructed with approximately 2 inches clear distance from the canopy structural steel to adjacent 1956+ Emergency Room Addition and 1987 Observation/Restroom Addition structures. Canopy finishes such as flashing and gutters have only approximately ½ inch clear distance to adjacent structures (see Photos 5 & 6 – Appendix C). Proximity to adjacent structure is less than the required 5.28 inches (4 percent of 11 foot canopy height), and is non-compliant.

RECOMMENDATION: Tier 2 analysis estimates deflections in excess of 2.19 inches (See Calculation Appendix F) and is non-compliant. Additional analysis may be performed to estimate

horizontal movement in a seismic event. Minor damage may occur due to pounding between structures during a seismic event. However, damage due to this condition is not anticipated to cause life safety structural concerns within the canopy or the adjacent buildings due to the canopy's light and flexible steel cantilevered column construction. Damages to canopy finishes and minor damages to the canopy's structural perimeter steel and adjacent building finishes are to be expected and could represent an obstruction to egress out of adjacent building exits. Egress issues are recommended to be further analyzed. Possible remediation of the hazard could be to install knee braces between the columns and beams above head clearance level to stiffen the canopy structure reducing expected deflections in a seismic event (see schematic retrofit detail **SSK-1** – Appendix G).

- b. SURFACE FAULT RUPTURE (ASCE Section A.6.1.3) – *“Surface fault rupture and surface displacement at the building site are not anticipated.”* The site is located in the Alquist-Priolo special study zone per the California Division of Mines and Geology (CDMG) Santa Rosa Quadrangle Map published in 1983. Multiple geologic hazard evaluations and geotechnical reports completed by five geotechnical firms were reviewed for this site. The oldest reviewed report was completed in 1978 while the most recent was completed in 2002. The exact location of faults in the area of the Chanate Hospital buildings is not clearly determined by the reports due to differing data and conclusions. The 2002 report was performed by Rutherford & Chekene (R&C) and included a geologic hazard evaluation by Gilpin Geosciences. Gilpin Geosciences Report summarizes the surface fault rupture hazard:

“Based on the preponderance of lineaments and other fault-related features observed by Gilpin Geosciences and others in the site vicinity, along with the lack of clear resolution of differing interpretations of onsite and offsite geologic structures, we conservatively judge the overall potential for fault rupture at the site to be high. There may exist areas within the site that are sufficiently free of active faults so as to allow future construction of structures for human occupancy.”

A lineament is a feature in a landscape which is an expression of an underlying geologic structure such as a fault. The 1978 Cooper Clark & Associates report estimated the maximum potential offset at the surface as 25 inches horizontally and 2.5 inches vertically. The report summarized the potential surface rupture behavior:

“If surface rupture were to occur along the part of the fault near the hospital, the displacement could occur along the relatively well located trace mapped to the west, along the approximately located trace mapped near this site, or along other traces, such as those found nearby in our trenches – a zone probably 980 feet wide.”

Based on the information contained in the reviewed reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred. Four of the five geotechnical engineers that provided information for the site concluded there were likely fault traces or fault related features extending through at least some portion of the site. Rutherford & Chekene's (R&C) conclusions for the site varied. In the 1986 report R&C concluded there were fault traces on the site but they are not considered active. Reports completed between 1987 and 1992 concluded there were no fault traces at the specific reviewed locations and the entire health care facility was suitable for development from a geologic/geotechnical viewpoint. The final R&C report in 2002 concluded it was likely that fault traces projected through the site and under the existing Medical Center complex. Additionally the report stated there was a high potential for surface rupture. There does appear to be areas where fault traces may be less likely to occur based on the fault maps provided. See the geotechnical summary for a complete discussion of all reviewed information pertaining to surface fault rupture.

The 1978 Cooper Clark & Associates report was the most extensive study of potential fault traces on the site that was reviewed. R&C performed many investigations with differing conclusions however the latest R&C report reviewed stated the overall potential for fault rupture at the site to

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be high. This finding is in line with Cooper Clark, Herzog and HLA (Herzog and HLA reports are for buildings not included in the scope of work for this project). The following is a summary of potential fault traces as it affects the building. See the Geotechnical Summary for a complete summary of all reviewed reports.

The 1978 Cooper Clark & Associates report projected two fault traces in the direction of the 1956 building. The 1986 R&C report found several fault traces that approximately aligned in location and orientation with the Cooper Clark report that would project beneath this building; however they classified the faults as non-active (more than 11,000 years old). The 2002 R&C report stated it is prudent to assume that active faults may project under the existing Medical Center complex.

RECOMMENDATION: Based on the information contained in the reviewed reports and the CDMG maps, the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. While both faults are clearly defined and located outside of the Santa Rosa area, the fault location at the hospital site is inferred.

Geologic records indicate that surface fault rupture and surface displacement at the building site are potentially anticipated. To determine fault rupture potential more conclusively, a comprehensive geologic and geotechnical review and investigation is recommended. This would include a review of all of the existing documentation, including contacting OSHPD regarding additional documentation in their files, as well as performing a current comprehensive site investigation with multiple fault exploration trenches located across the site. For additional information, see the geotechnical and geologic summary for the site.

Based on the relative small size and value of this structure, relocation of the building is not likely a reasonable solution. Remediation of this deficiency is likely not feasible. Based on small size of the canopy, existing seismic separations between adjacent buildings and the flexibility of cantilevered column systems, minor fault offsets are not likely to cause collapse. Large offsets near the maximum possible could cause localized failures or loss of bearing; however, the structure overall is not likely to collapse. Significant fault rupture within the canopy envelope is likely to damage the canopy beyond repair or future use.

An existing building with surface fault rupture potential is not specifically prohibited from occupancy by the California Building Code or the Alquist-Priolo Act. However, the inherent risks associated with seismic activity are increased.

- c. **COMPACT MEMBERS** (ASCE Section A.3.1.3.8) – *“All frame elements meet section requirements set forth by AISC 341 Table D1.1, for moderately ductile members.”* The TS6x6x3/16 cantilevered columns have a b/t ratio of 31.5 exceeding the Table D1.1 moderately ductile limit of 17.9 for 37 ksi steel tubes and are non-compliant.

Tier 2 analysis indicates TS6x6x3/16 cantilevered columns are adequate for the required loading to meet Life Safety standards (See Appendix F). **DEFICIENCY WAIVED.**

RELIABILITY OF SEISMIC EVALUATIONS

In general, structural engineers do not have the ability to predict the exact damage to a building as a result of an earthquake. There will be a wide variation of damage from building to building due to the variations in ground motion and varying types and quality of construction. In addition, engineers cannot predict the exact ground motions of the earthquake that may strike a given building. Design and evaluation of buildings are performed using general guidelines and information from past earthquakes. Engineers and the codes used for design and evaluation have been conservative when attempting to ensure that building design meets minimum standards of life safety. This effort is based on science and technology as well as on observations made from actual seismic events. Building design and evaluation codes are constantly evolving to better meet performance targets based on this information. Continued research will improve predictive methods and facilitate performance-based engineering. It has been estimated that, given design ground motions, a small percent of new buildings and a slightly greater percent of retrofit buildings may fail to meet their expected performance.

CLOSING

The seismic review and analyses associated with this evaluation were based on available original structural drawings, and the site reviews were based on that which was plainly visible. No attempt was made to uncover hidden conditions or perform any destructive or non-destructive testing. The items discussed in this report are subject to revision should more information become available.

This report is general in nature and does not imply that the recommendations listed above are the only structural requirements that must be made to the existing structure to meet current code criteria.

We understand you may have questions regarding this evaluation and are available for comment and explanations. Please call with any questions you may have. Thank you for choosing ZFA Structural Engineers to assist you with this building seismic review.

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APPENDIX A – GEOTECHNICAL AND GEOLOGIC SUMMARY

Geotechnical and Geologic Summary

The site is located in the Alquist-Priolo special study zone per the California Division of Mines and Geology Santa Rosa Quadrangle Map published in 1983. The Alquist-Priolo act's main purpose is to prevent the construction of buildings for human occupancy on the surface trace of active faults and requires the following for new construction on sites located within the special study zone:

“Before a project can be permitted, cities and counties must require a geologic investigation to demonstrate that proposed buildings will not be constructed across active faults. An evaluation and written report of a specific site must be prepared by a licensed geologist. If an active fault is found, a structure for human occupancy cannot be placed over the trace of the fault and must be set back from the fault (generally 50 feet).”

Geologic Hazard Evaluations and Geotechnical Reports were provided and reviewed for this report. 5 different geotechnical Engineers provided reports that were reviewed including: Cooper Clark & Associates - 1978 geologic hazard report, Rutherford & Chekene Consulting Engineers (R&C) - 1986 geologic hazard report, Harding and Lawson - 1986 Family Practice Clinic geotechnical report, R&C 1987 - Emergency Room Expansion geotechnical report, R&C – 1988 Family Practice Clinic Geotechnical report, R&C – 1988 Power Plant geotechnical report, R&C – 1990 Medical Office Building geotechnical report, Herzog – 1991 & 1992 review letters of R&C Medical Office Building geotechnical report, R&C 1991 & 1992 responses to Herzog review letters, R&C 2002 Geologic and Seismic Hazard Evaluation and geotechnical study and the geologic hazard portion of this report was performed by Gilpin Geosciences (appendix to R&C 2002 report).

In 1978 Cooper Clark performed a Geo-Hazard Evaluation for the northern portion of the site which including eight trenches with average depths of 5 to 10 feet and a maximum depth of about 12 feet. This report also referenced results from three additional trenches completed by Chanate Corp. but no trench information was provided. Cooper Clark located what they determined to be multiple fault traces on the site including two traces that projected below the 1972 building. If the fault traces continued on the same orientation it appeared the traces would extend below the western side of the 1956 and 1936 buildings. Cooper Clark estimated a maximum anticipated surface offset of 25” horizontally and 3” vertically. The report also stated that since many offsets were found in the small area investigated suggests that ruptures may occur at many places within the weak upper materials in the wide fault zone rather than along a few well defined narrow traces.

In 1986 R&C completed a Geo-Hazard Evaluation including three trenches with depths ranging from 6 to 10 feet and six ground magnetic and refraction seismic surveys mostly concentrated on the north east corner of the site. R&C found several fault traces in the trenches that appear to align, both in location and in orientation with the fault traces found in the 1978 Cooper Clark report. R&C concluded the faults were not active because they didn't project through material that was less than 11,000 years old (no offset of material for 11,000 years plus).

In 1986 Harding Lawson Associates (HLA) preformed a fault trace investigation including two trenches with an average depth of 6 feet for a proposed addition to the Family Practice Clinic. The site was located on the south side of Chanate and west of the primary entrance to the hospital campus. They found two fault traces that projected through the planned addition as well as the west end of the existing structure. HLA proposed a 20 foot setback for future structures on the site. The proposed building was not built.

In 1987 R&C completed a geotechnical study for a proposed emergency room extension between the 1972 and 1956 buildings which included two trenches with an average depth of 4 or 5 feet in the proposed footprint of the building. These trenches did not extend into bedrock and R&C found no evidence of fractured bedrock reviewing the top of the rock. Based on the results of this investigation, a reevaluation of the R&C 1986 Geologic Hazard Investigation as well as regional reconnaissance performed for the report, R&C concluded that the proposed building site as well as the entire Health Facility was deemed suitable for development from a geological/geotechnical viewpoint.

In 1988 R&C completed a geotechnical study for a revised Family Practices Clinic Addition including one trench with an average depth of 10 feet. R&C trench was located approximately 20 feet uphill and parallel to the 1986

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HLA trenches. R&C appeared to find similar geological features as HLA however; R&C concluded these features were conformable sedimentary contacts. Additionally the report reiterated R&C's conclusion from the 1987 Geologic Hazard Report that the entire Healthcare Facility was suitable for development. This building was constructed but the building location on the site appears to conform to the recommended set back from the 1986 HLA report.

In 1990 R&C completed a geotechnical study for a proposed medical office building including one trench with an average depth of 5 feet. The new building site was located south of Chanate Road and approximately in line with the 1936, 1956 and 1972 buildings. The original report concluded, referencing 1987 Geologic Hazard Report, that the site was suitable for the proposed building. The geotechnical report for the medical office building was peer reviewed by Herzog. The scope of Herzog's review was to provide an opinion as to whether R&C's report constituted a fault evaluation report that is in conformance with the policies and criteria of the CDMG special publication 42. Over the process, approximately 2 years, Herzog's scope increased to include whether R&C supplementary work was responsive to the plan of work agreed to in meetings with the City of Santa Rosa. The entire correspondence between R&C and Herzog was not provided for review, however it appears Herzog had multiple concerns with R&C initial report including, aerial photographs used, illustrations including comments on the trench log, clarification of fault creep information provided and the trench wall cleaning and logging procedure. Through several correspondences and at least one meeting with the City of Santa Rosa it was determined a new trench should be performed. The second trench was completed in December 1991 which was reviewed by R&C, Herzog, a representative from the California Division of Mines and Geology (CDMG) and a representative from the City of Santa Rosa. Herzog, with reference to CDMG, concluded there were several features in the trench that were fault-related structures while R&C continued to state the site was acceptable to build on. ZFA does not possess documentation that contains the conclusion of the discussion relating to possible fault traces below the proposed medical office building. The building was not constructed.

In 2002 R&C again performed a geologic and seismic hazard evaluation for the entire site. For this report R&C hired Gilpin Geosciences to perform the geologic hazard evaluation portion of the report. Gilpin reviewed all of the above documents plus several additional documents that ZFA does not have copies of. No additional trenching was completed at this time. Gilpin Geosciences Report summarized the findings:

"Based on the preponderance of lineaments and other fault-related features observed by Gilpin Geosciences and others in the site vicinity, along with the lack of clear resolution of differing interpretations of onsite and offsite geologic structures, we conservatively judge the overall potential for fault rupture at the site to be high. There may exist areas within the site that are sufficiently free of active faults so as to allow future construction of structures for human occupancy."

R&C restated this conclusion in their report. This represents a change in conclusion from R&C previous statements regarding the entire site being free of faults. This is the last document ZFA reviewed from R&C and assumes it supersedes the previously issued R&C conclusions. Additionally the 2002 R&C report included a similar conclusion to the 1978 Cooper Clark report regarding the complexity of the fault structure in the site and the potential for fault rupture that triggers movement on discontinuous subsidiary structures and sympathetic small movements on many fractures across the entire fault zone.

See the Findings and Recommendations section for each building for a detailed explanation of potential fault traces as they relate to each building.

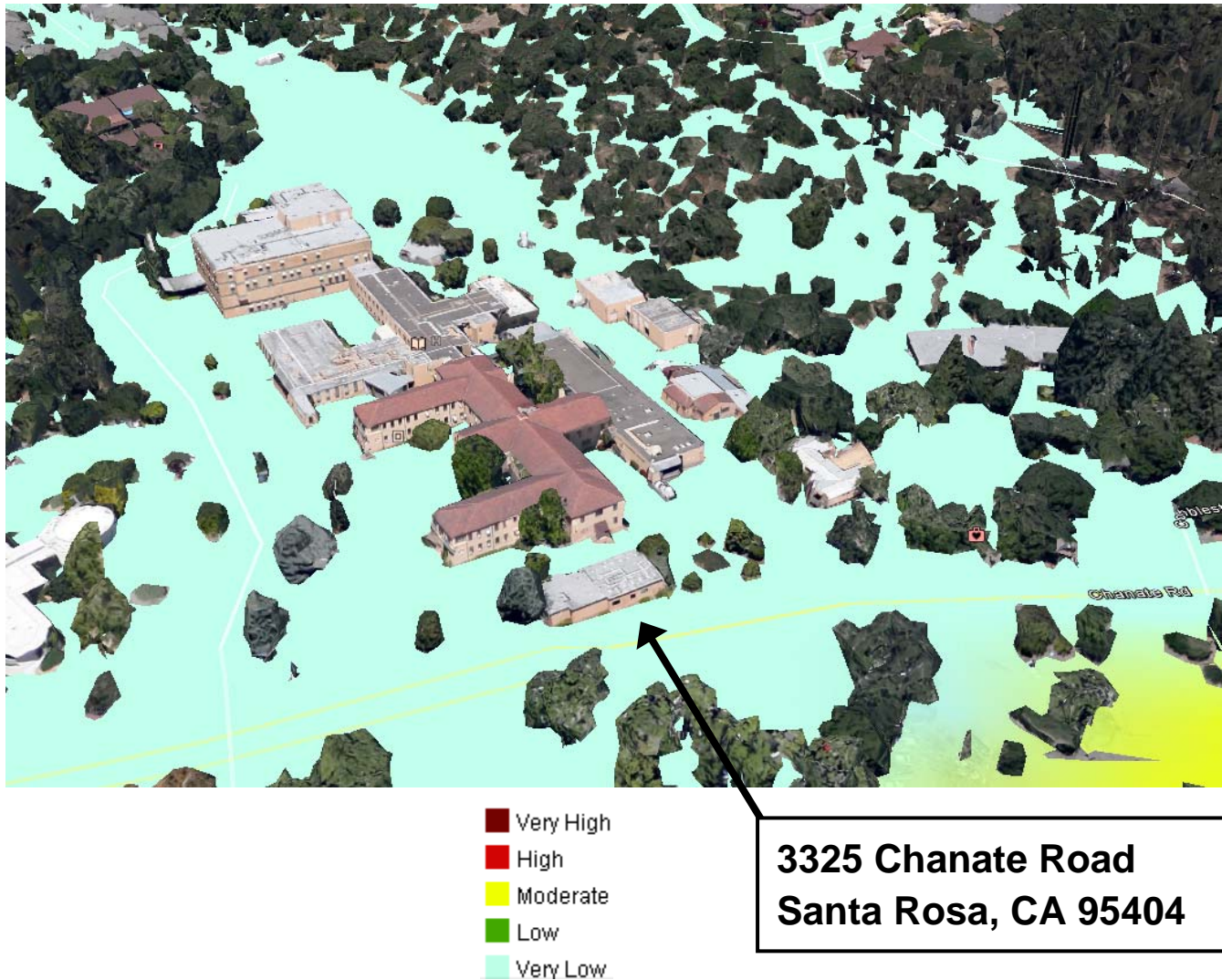
Based on the information contained in the geotechnical reports and the CDMG maps the intersection of the Rodgers Creek Fault, to the south, and the Healdsburg Fault, to the north, is projected to occur in the area of the hospital site. The reviewed documents indicate that surface fault rupture and surface displacement at the site are potentially anticipated however, this displacement may occur across a complex series of faults.

APPENDIX B – MAPS

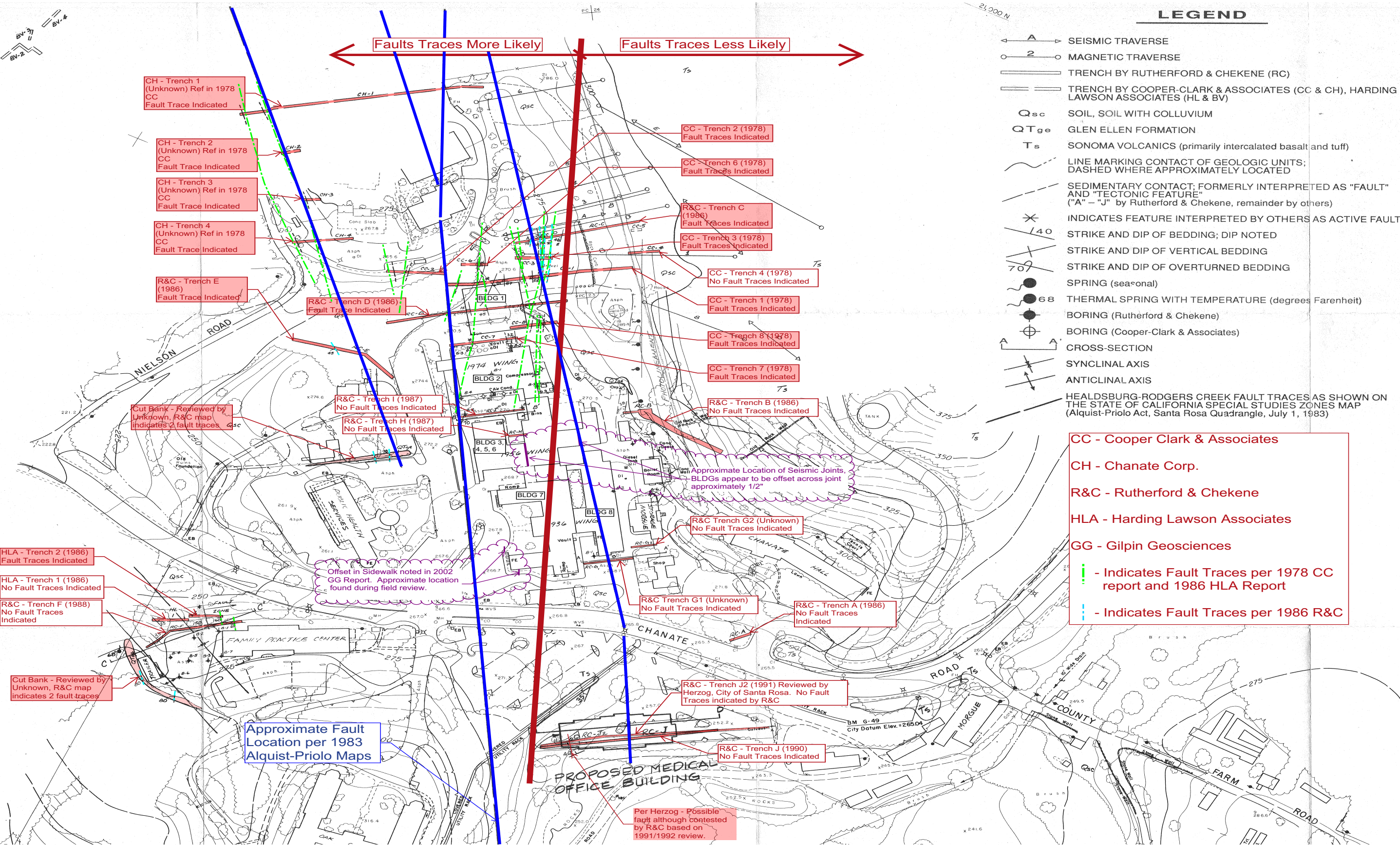
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Liquefaction Susceptibility Map

Per the ABAG Liquefaction Susceptibility Map below, the subject site is located in an area that has very low probability for liquefaction in a seismic event.

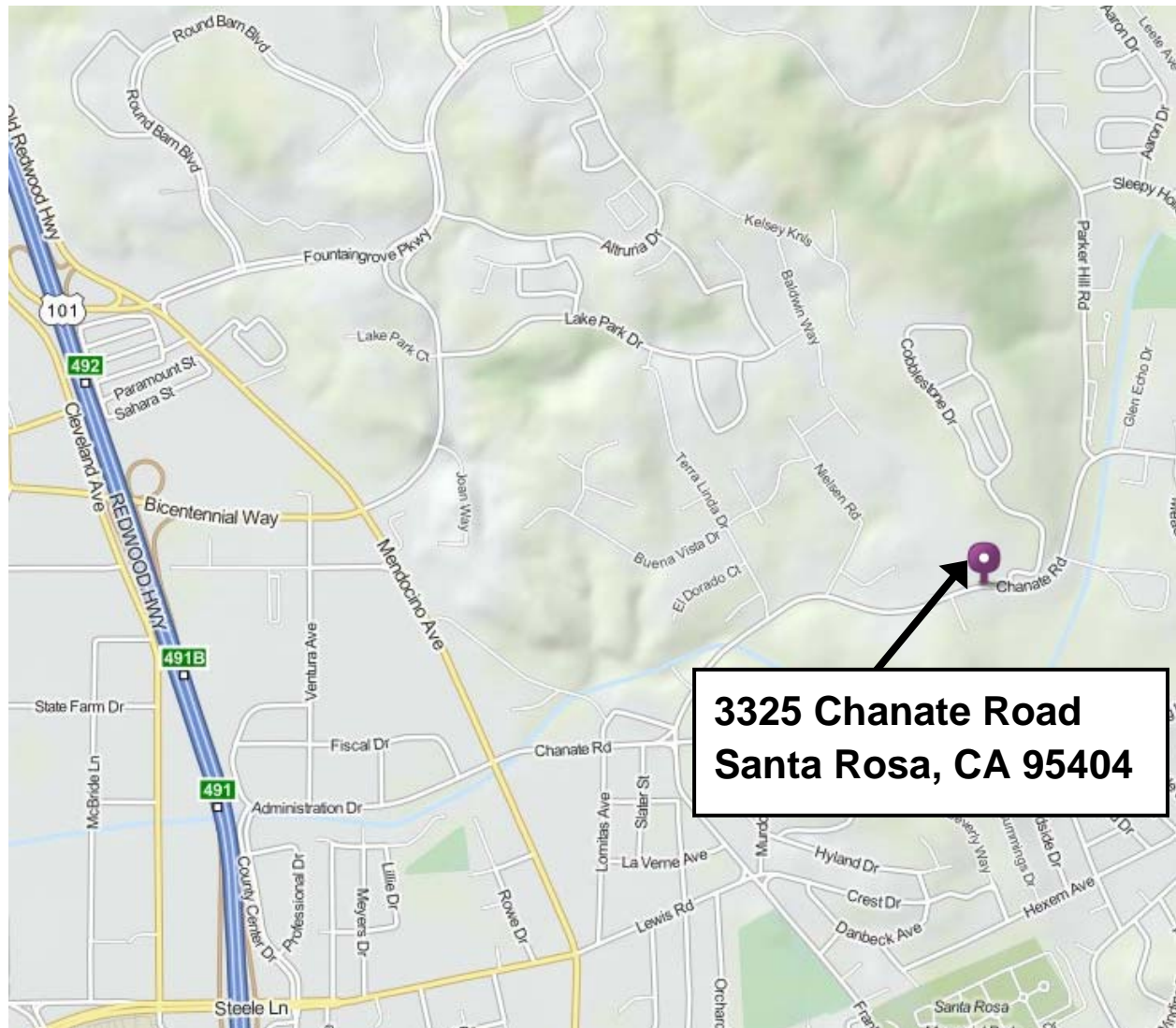


Geotechnical Fault Trace Map



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Location Map

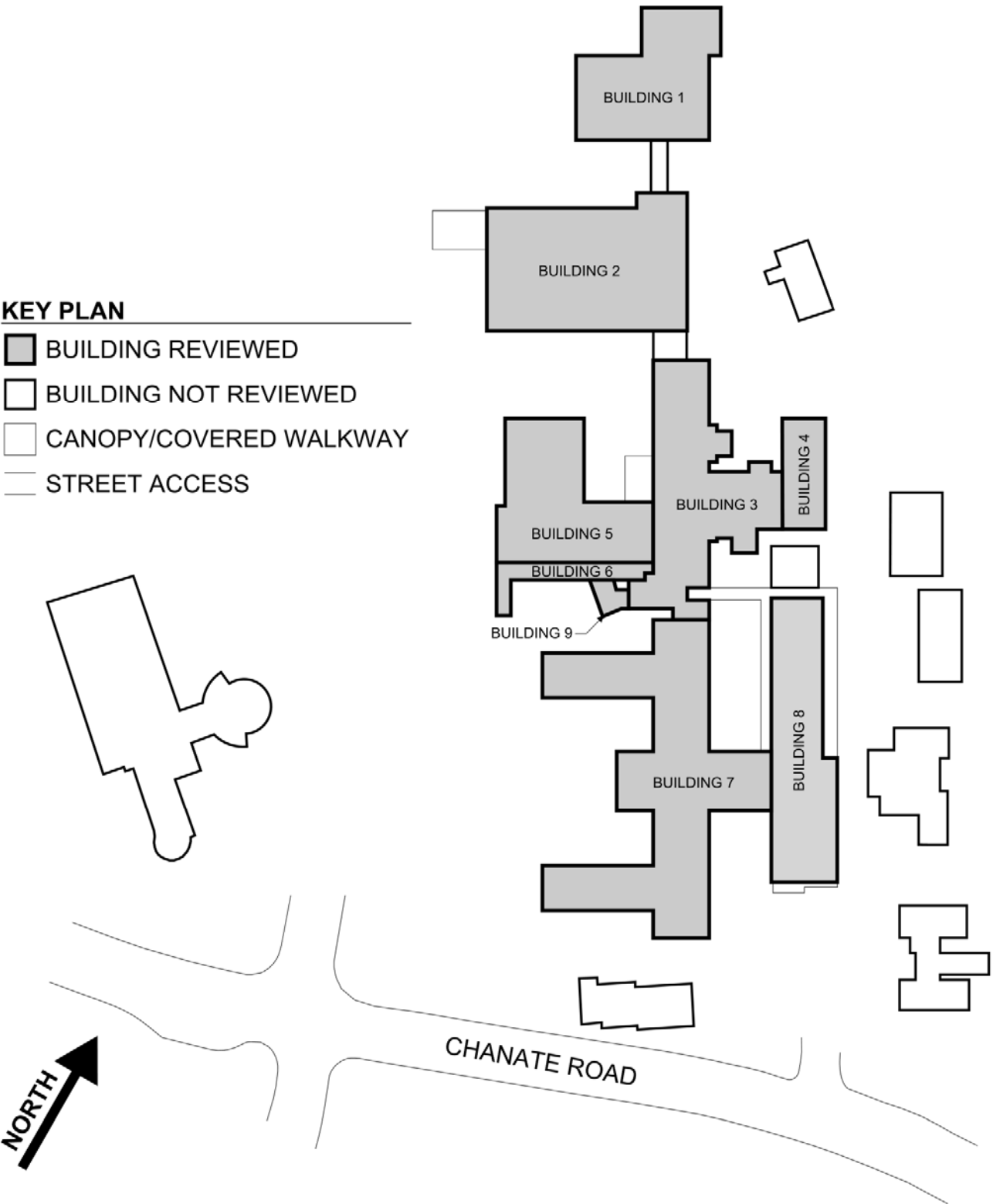


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Schematic Site Plan

KEY PLAN

-  BUILDING REVIEWED
-  BUILDING NOT REVIEWED
-  CANOPY/COVERED WALKWAY
-  STREET ACCESS



APPENDIX C – PHOTOGRAPHS

3325 Chanate Road, Santa Rosa, CA 95404

Building 1 (1999-2004 Cath Lab)



Photo 1: Building 1c: 2004 Cath Lab Addition – West Elevation



Photo 2: Building 1a: 1999 Cath Lab – North Elevation

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Photo 3: Buildings 1a and 1b East Elevation

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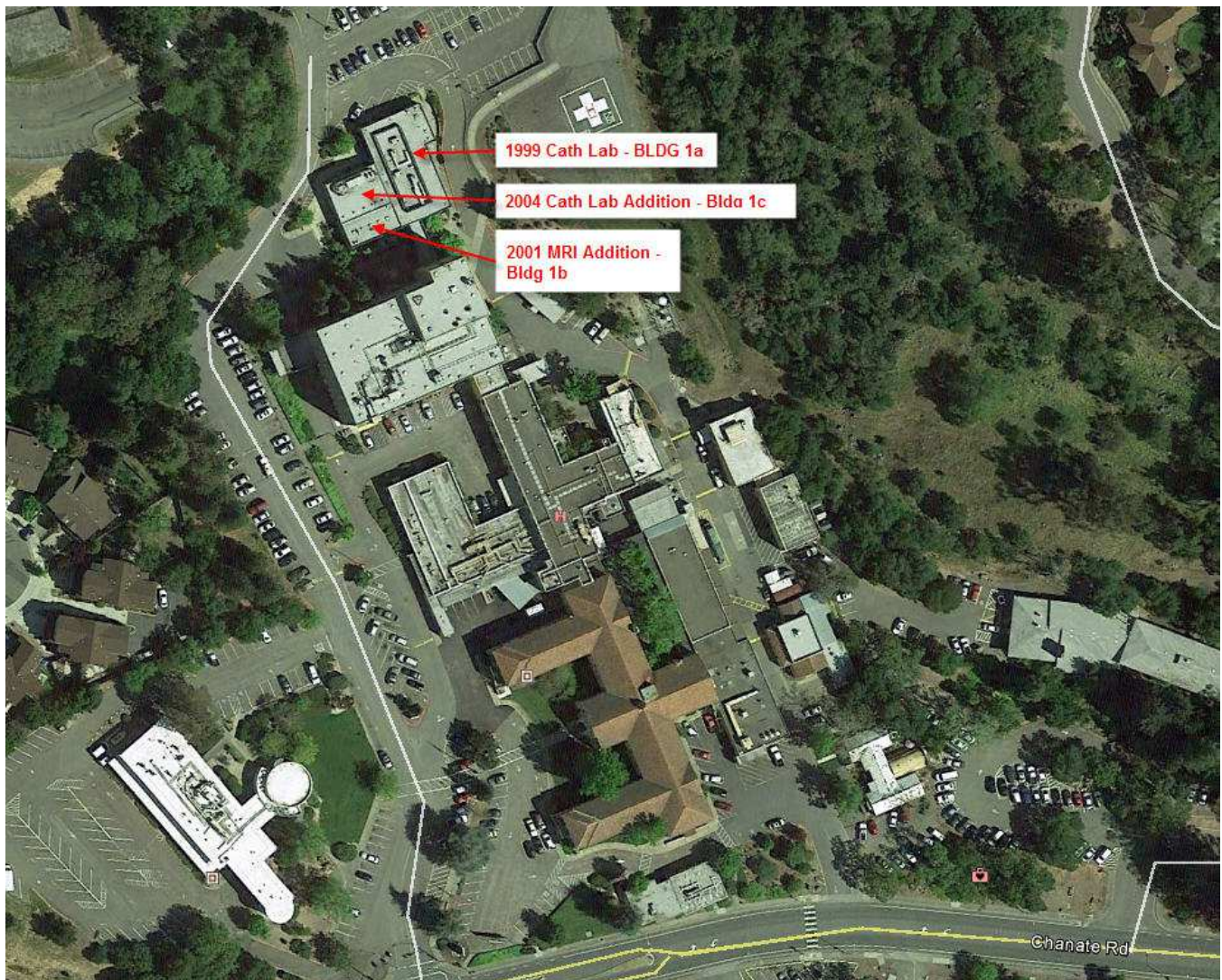


Photo 4: Site Plan

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Building 2 (1972 Acute Care Hospital)



Photo 1: Building 2 – Northwest Corner



Photo 2: Building 2 – Northeast Corner

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Photo 3: Building 2 – Southwest Corner



Photo 4: Building 2 – South Elevation

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Building 3 (1956 Hospital Wing)



Photo 1: Building 3 – Southwest Corner



Photo 2: Building 3 – Northwest Corner

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Building 4 (1956 Hospital Wing)



Photo 3: Building 4 – Northeast Corner



Photo 4: Building 4 – Northwest Corner

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Building 5 (1956 Hospital Wing)



Photo 5: Building 5 – Northwest Corner



Photo 6: Building 5 – Southwest Corner (middle) and Building 6 – Telecommunications Addition (right)

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Building 6 (1956 Hospital Wing)



Photo 7: Building 6 – Southwest Elevation



Photo 8: Building 6 – Southeast Elevation

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Building 3 Appendages (1956 Hospital Wing)



Photo 9: Building 3 – Steel Framed 1-Story “Area A” Appendage



Photo 10: Building 3 – Steel Framed 2-Story “Area B” Appendage (Reduced from Plans)

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Photo 11: Building 3 – Steel Framed 2-Story “Area C” Appendage



Photo 12: Building 3 – Steel Framed 1-Story “Area D” Appendage

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Photo 13: Building 6 – Rust signs at steel ledger

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Building 7 (1936 Original Hospital Building)



Photo 1: Building 7 – Main Entry at West Central Projection



Photo 2: Building 7 – Southeast Location at Unbuilt Wing



Photo 3: Roof Joist at Exterior Wall (note the lack of shear blocking and reliance on weak axis bending of thin gage truss heel; note minimal track splice at right)

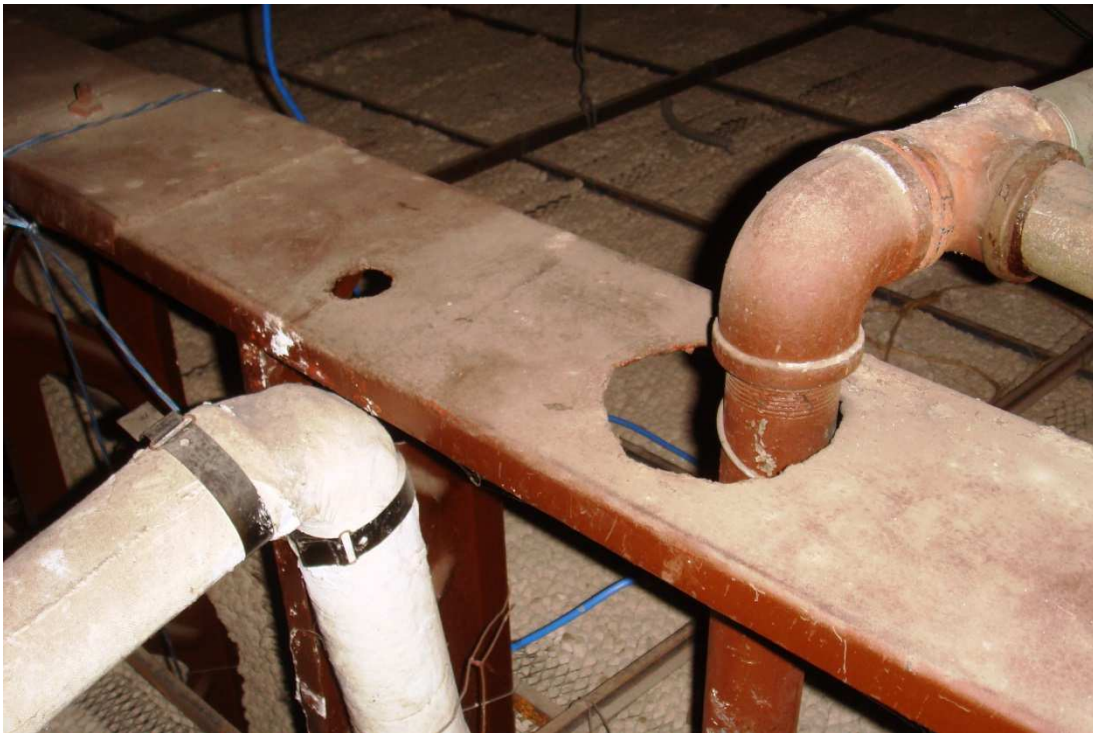


Photo 4: Interior Braced Wall Panel above Ceiling (note large pipe penetrations and minimal track splice at left)



Photo 5: Interior Braced Wall Panel End Connection above Ceiling (note eccentricity and tack weld connections)

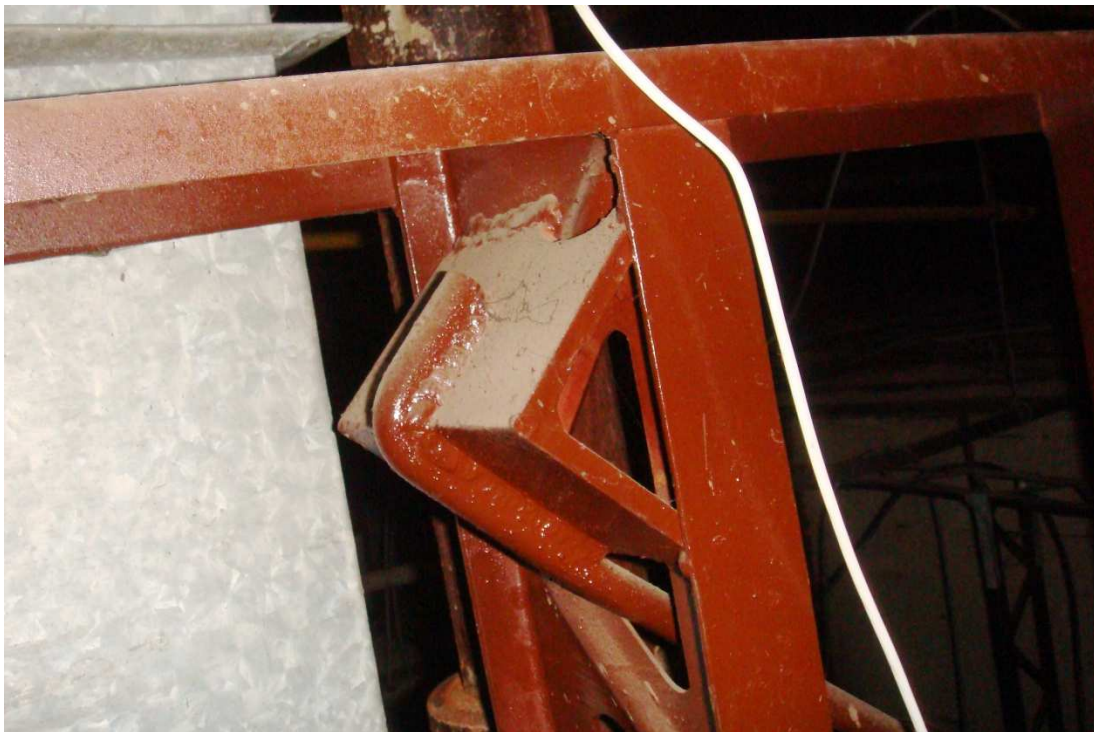


Photo 6: Interior Braced Wall Panel End Connection above Ceiling (note eccentricity and tack weld connections)



Photo 7: Interior Braced Wall Panel End Connection above Ceiling (note missing nut)



Photo 8: Diagonal Rod Splice (note use of rebar and eccentric lap splice)



Photo 9: Floor Joist at Corridor Wall (note lack of shear blocking at cripple studs and discontinuous diaphragm at full height wall studs; note minimal angle ledger with minimal splice at the left)

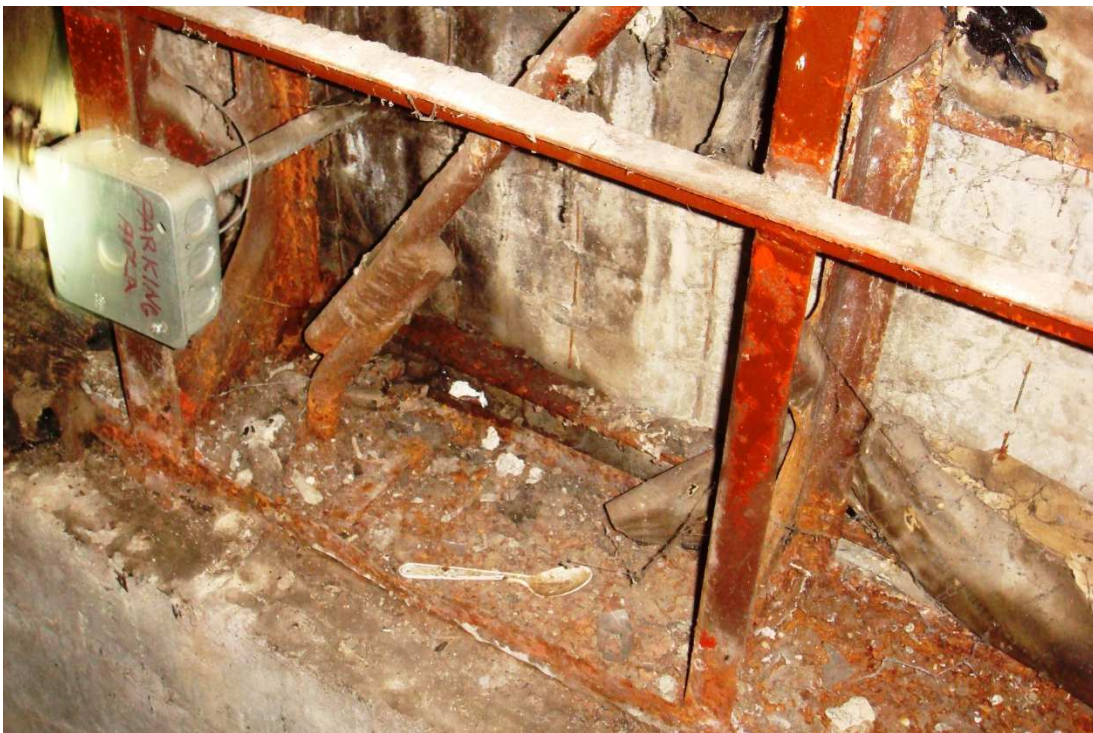


Photo 10: Diagonal Rod at Basement Wall (note bent anchor stub at the left and sill track corrosion)



Photo 11: Missing Rod at Basement Wall (note unused anchor stub at the left and sill track corrosion)



Photo 12: Cripple Studs at Basement Wall (note cut and bent stud at the left and corrosion)

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Building 8 (1956 Kitchen/Storage Building)



Photo 1: Building 8 – South Exterior



Photo 2: Building 8 – East Covered Loading Dock

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Photo 3: Building 8 – West Covered Walkway



Photo 4: Building 8 Roof and Adjacent 1936 Building

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Building 9 (1987 Ambulance Canopy)



Photo 1: Building 9 – Southwest Elevation

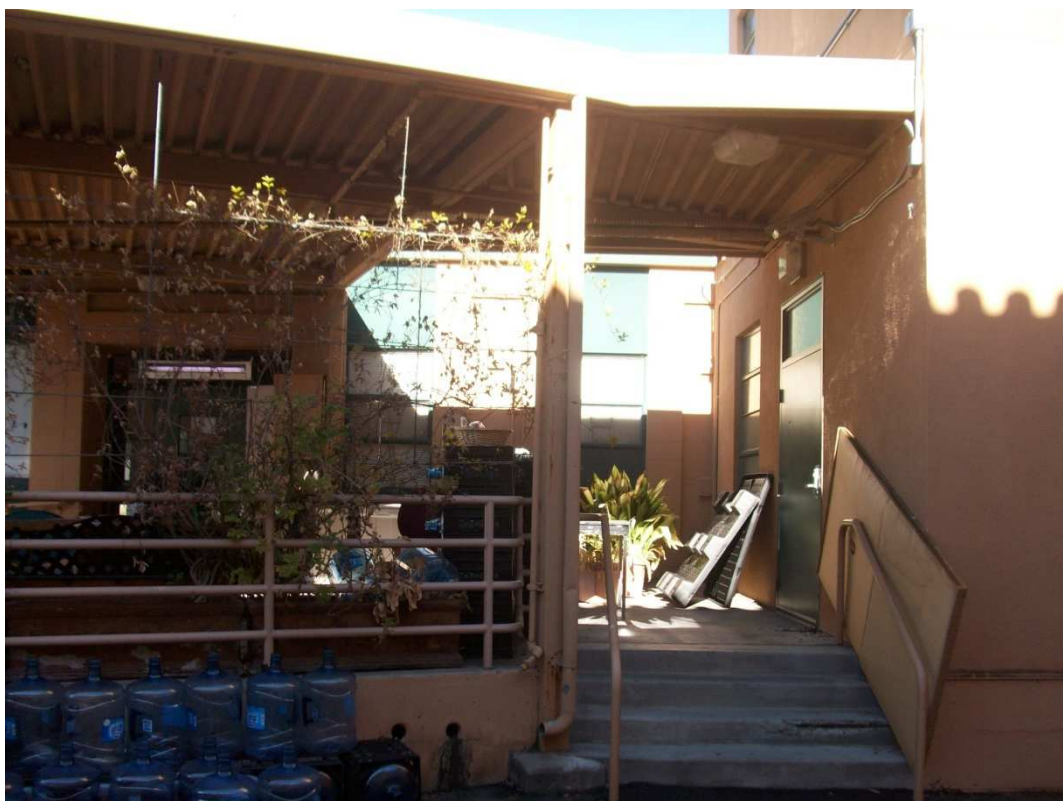


Photo 2: Building 9 – Southeast Elevation



Photo 3: Building 9 – Cantilevered Column Top Connection



Photo 4: Building 9 – Cantilevered Column Fixed (Embedded) Base Connection



Photo 5: Building 9 – Northwest Seismic Gap



Photo 6: Building 9 – Northeast Seismic Gap

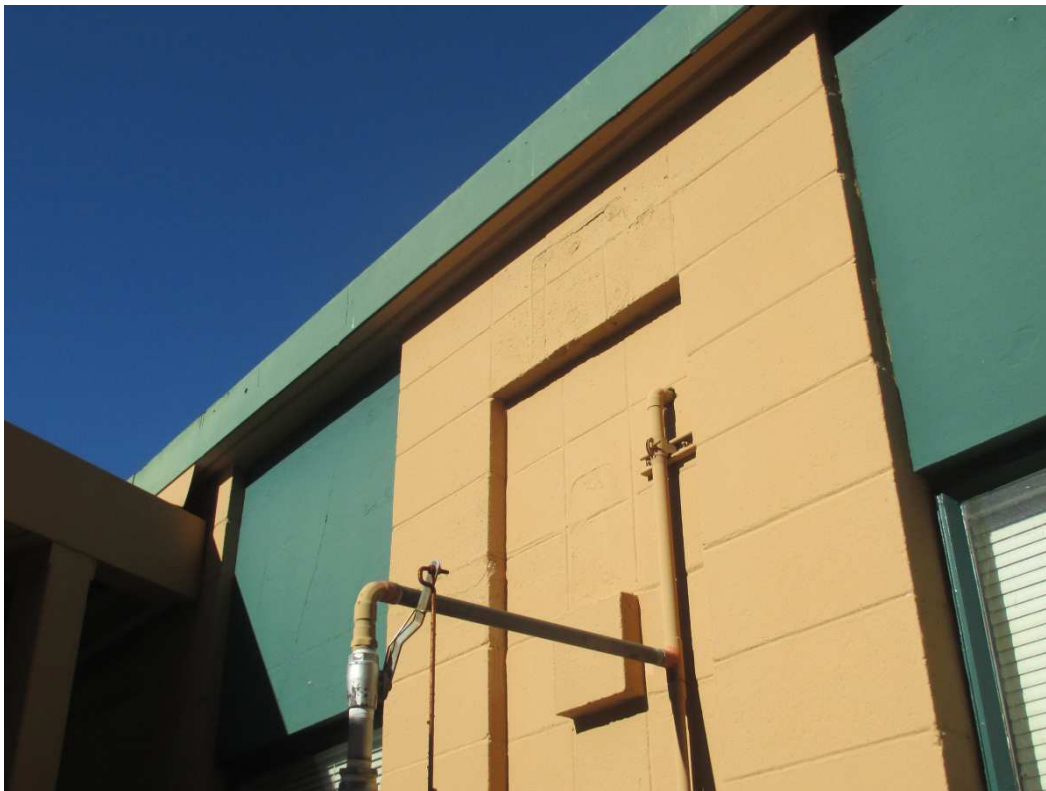


Photo 7: Building 9 – 1973± Canopy Connection Removed



Photo 8: Building 9 – 1973± Canopy Connection Removed

APPENDIX D – SUMMARY DATA SHEET AND MATERIAL PROPERTIES

3325 Chanate Road, Santa Rosa, CA 95404

Building 1a (1999-2004 Cath Lab) Summary Data Sheet**BUILDING DATA**

Building Name: 1999 Cath Lab Addition – Building 1a

Date: 11/7/2014

Building Address: 3325 Chanate Rd Santa Rosa, CA 95404

Latitude: 38.47037° N

Longitude: 122.70816° W

By:

Year Built: 1999

Year(s) Remodeled:

Original Design Code: 1997 UBC

Area (sf): 4916

Length (ft): 95'-0"

Width (ft): 58'-0"

No. of Stories: 1

Story Height: 14'-0"

Total Height: 14'-0"

USE ☐ Industrial ☐ Office ☐ Warehouse ☒ Hospital ☐ Residential ☐ Educational ☐ Other:**CONSTRUCTION DATA**

Gravity Load Structural System: Steel Stud joists and Wood Framed walls

Exterior Transverse Walls: Wood studs

Openings? Yes

Exterior Longitudinal Walls: Wood studs

Openings? Yes

Roof Materials/Framing: Modified Bitumen w/ Structural wood sheathing over 8" Steel Joists at 24" oc

Intermediate Floors/Framing: None

Ground Floor: 12" Steel Joists over crawl space

Columns: Steel Columns at Main Roof Girders

Foundation: Shallow concrete spread footings

General Condition of Structure: Appears to be in generally good structural condition

Levels Below Grade? None

Special Features and Comments: A Seismically Isolated steel framed pedestrian walkway was built during the 1999 Cath Lab addition to connect to Building 2 but was not reviewed.

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Wood Shear walls w/ Structural sheathing	Wood Shear walls w/ Structural sheathing
Vertical Elements:	Wood Shear walls w/ Structural sheathing	Wood Shear walls w/ Structural sheathing
Diaphragms:	Wood structural sheathing	Wood structural sheathing
Connections:	Nailing and metal hardware	Nailing and metal hardware

EVALUATION DATA

BSE-1N Spectral Response Accelerations:

 $S_{DS} = 1.66$ $S_{D1} = 0.891$

Soil Factors:

Class= C

 $F_a = 1.0$ $F_v = 1.3$

BSE-1E Spectral Response Accelerations:

 $S_{XS} = 0.996$ $S_{X1} = 0.548$

Level of Seismicity:

SDC-E (HIGH)

Performance Level: Life Safety (S-3)

Building Period:

 $T = 0.145$ s

Spectral Acceleration:

 $S_a = 0.996$

Modification Factor:

 $C_m C_1 C_2 = 1.3$ Building Weight: $W = 177,000$ lbs

Pseudo Lateral Force:

 $V = C_m C_1 C_2 S_a W = 229,180$ lbs

BUILDING CLASSIFICATION: W2 – Wood Frames, Commercial and Industrial

REQUIRED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Building Type W2 Structural Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Nonstructural Component Checklist	<input type="checkbox"/>	<input checked="" type="checkbox"/>

FURTHER EVALUATION REQUIREMENT: No

3325 Chanate Road, Santa Rosa, CA 95404

Building 1b (1999-2004 Cath Lab) Summary Data Sheet**BUILDING DATA**

Building Name:	2001 MRI Addition-Building 1b		Date:	11/7/2014
Building Address:	3325 Chanate Rd Santa Rosa, CA 95404			
Latitude:	38.47037° N	Longitude:	122.70816° W	By: CSB
Year Built:	2001	Year(s) Remodeled:		Original Design Code:
Area (sf):	1138	Length (ft):	48'-0"	Width (ft):
No. of Stories:	1	Story Height:	14'-0"	Total Height:
				14'-0"

USE ☐ Industrial ☐ Office ☐ Warehouse ☒ Hospital ☐ Residential ☐ Educational ☐ Other:

CONSTRUCTION DATA

Gravity Load Structural System:	Wood Framed		
Exterior Transverse Walls:	Wood studs	Openings?	Yes
Exterior Longitudinal Walls:	Wood studs	Openings?	Yes
Roof Materials/Framing:	Comp Roof w/ Structural wood sheathing over I joist at 16" oc.		
Intermediate Floors/Framing:	None		
Ground Floor:	Concrete Slab on Grade		
Columns:	None	Foundation:	Shallow concrete spread footings
General Condition of Structure:	Appears to be in generally good structural condition		
Levels Below Grade?	None		
Special Features and Comments:			

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Wood Shear walls w/ Structural sheathing	Wood Shear walls w/ Structural sheathing
Vertical Elements:	Wood Shear walls w/ Structural sheathing	Wood Shear walls w/ Structural sheathing
Diaphragms:	Wood structural sheathing	Wood structural sheathing
Connections:	Nailing and metal hardware	Nailing and metal hardware

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	$S_{DS} =$	1.66	$S_{D1} =$	0.891
Soil Factors:	Class =	C	$F_a =$	1.0
			$F_v =$	1.3
BSE-1E Spectral Response Accelerations:	$S_{XS} =$	0.996	$S_{X1} =$	0.548
Level of Seismicity:	SDC-E (HIGH)	Performance Level:	Life Safety (S-3)	
Building Period:	T =	0.145 s		
Spectral Acceleration:	$S_a =$	0.996		
Modification Factor:	$C_m C_1 C_2 =$	1.3	Building Weight: W =	37,730 lbs
Pseudo Lateral Force:	$V = C_m C_1 C_2 S_a W =$	48,853 lbs		

3325 Chanate Road, Santa Rosa, CA 95404

BUILDING CLASSIFICATION: W2 – Wood Frames, Commercial and Industrial

REQUIRED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Building Type W2 Structural Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Nonstructural Component Checklist	<input type="checkbox"/>	<input checked="" type="checkbox"/>

FURTHER EVALUATION REQUIREMENT: No

3325 Chanate Road, Santa Rosa, CA 95404

Building 1c (1999-2004 Cath Lab) Summary Data Sheet**BUILDING DATA**

Building Name:	2004 Cath Lab Addition-Building 1c		Date:	11/7/2014
Building Address:	3325 Chanate Rd Santa Rosa, CA 95404			
Latitude:	38.47037° N	Longitude:	122.70816° W	By: CSB
Year Built:	2004	Year(s) Remodeled:		Original Design Code:
Area (sf):	1862	Length (ft):	48'-0"	Width (ft):
No. of Stories:	1	Story Height:	14'-0"	Total Height:
				14'-0"

USE ☐ Industrial ☐ Office ☐ Warehouse ☒ Hospital ☐ Residential ☐ Educational ☐ Other:

CONSTRUCTION DATA

Gravity Load Structural System:	Wood Framed		
Exterior Transverse Walls:	Wood studs	Openings?	Yes
Exterior Longitudinal Walls:	Wood studs	Openings?	Yes
Roof Materials/Framing:	SBS Mod Bitumen roof w/ Structural wood sheathing over I joist at 24" oc.		
Intermediate Floors/Framing:	None		
Ground Floor:	Concrete Slab on Grade		
Columns:	Steel Hollow Structural sections (HSS)	Foundation:	Shallow concrete spread footings
General Condition of Structure:	Appears to be in generally good structural condition		
Levels Below Grade?	None		
Special Features and Comments:			

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Wood Shear walls w/ Structural sheathing	Wood Shear walls w/ Structural sheathing
Vertical Elements:	Wood Shear walls w/ Structural sheathing	Wood Shear walls w/ Structural sheathing
Diaphragms:	Wood structural sheathing	Wood structural sheathing
Connections:	Nailing and metal hardware	Nailing and metal hardware

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	$S_{DS} =$	1.66	$S_{D1} =$	0.891
Soil Factors:	Class =	C	$F_a =$	1.0 $F_v =$ 1.3
BSE-1E Spectral Response Accelerations:	$S_{XS} =$	0.996	$S_{X1} =$	0.548
Level of Seismicity:	SDC-E (HIGH)	Performance Level:	Life Safety (S-3)	
Building Period:	$T =$	0.145 s		
Spectral Acceleration:	$S_a =$	0.996		
Modification Factor:	$C_m C_1 C_2 =$	1.3	Building Weight: $W =$	70,309 lbs
Pseudo Lateral Force:	$V = C_m C_1 C_2 S_a W =$	91,402 lbs		

3325 Chanate Road, Santa Rosa, CA 95404

BUILDING CLASSIFICATION: W2 – Wood Frames, Commercial and Industrial

REQUIRED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Building Type W2 Structural Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Nonstructural Component Checklist	<input type="checkbox"/>	<input checked="" type="checkbox"/>

FURTHER EVALUATION REQUIREMENT: No

3325 Chanate Road, Santa Rosa, CA 95404

Building 2 (1972 Acute Care Hospital) Summary Data Sheet**BUILDING DATA**

Building Name:	1972 Wing Sutter Hospital – Building 2		Date:	11/7/14	
Building Address:	3325 Chanate Road, Santa Rosa, CA 95404				
Latitude:	38.470053	Longitude:	-122.708156	By:	AIZ
Year Built:	1972	Year(s) Remodeled:	Unknown	Original Design Code:	1970 UBC Assumed
Area (sf):	56,000 +/-	Length (ft):	144'	Width (ft):	90'
No. of Stories:	4 + penthouse	Story Height:	12'-6"	Total Height:	61'-4"

USE ☐ Industrial ☐ Office ☐ Warehouse ☒ Hospital ☐ Residential ☐ Educational ☒ Other: Currently Vacant

CONSTRUCTION DATA

Gravity Load Structural System:	Steel Frame		
Exterior Transverse Walls:	Steel Frame	Openings?	N/A
Exterior Longitudinal Walls:	Steel Frame	Openings?	N/A
Roof Materials/Framing:	Concrete over metal decking		
Intermediate Floors/Framing:	Concrete over metal decking		
Ground Floor:	Slab on grade		
Columns:	Steel	Foundation:	Spread and Strip
General Condition of Structure:	Good		
Levels Below Grade?	None		
Special Features and Comments:	Pre-1972 Code Moment Frame Connections (Non-FEMA 267-350)		

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Perimeter Steel Moment Frame	Perimeter Steel Moment Frame
Vertical Elements:	Steel Wide Flange Columns	Steel Wide Flange Columns
Diaphragms:	Concrete over metal decking	Concrete over metal decking
Connections:	Pre-1972 Moment Frame	Pre-1972 Moment Frame

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	$S_{DS} = 1.65\text{ g}$	$S_{D1} = 0.892\text{ g}$
Soil Factors:	Class= C	$F_a = 1.0$ $F_v = 1.3$
BSE-1E Spectral Response Accelerations:	$S_{XS} = 0.996\text{ g}$	$S_{X1} = 0.548\text{ g}$
Level of Seismicity:	High	Performance Level: Life Safety
Building Period:	$T = 0.942\text{ sec}$	
Spectral Acceleration:	$S_a = 0.58\text{ g}$	
Modification Factor:	$C_m C_1 C_2 = 0.9 * [1.2] = 1.08$	Building Weight: $W = 5,332\text{ kips}$
Pseudo Lateral Force:	$V = C_m C_1 C_2 S_a W = 1.08 * 0.58 * 5332 = 3,340\text{ kips}$	

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BUILDING CLASSIFICATION: S1 – Steel Moment Frame w/ Stiff Diaphragms**REQUIRED TIER 1 CHECKLISTS**

	Yes	No
Basic Configuration Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Building Type S1 Structural Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Nonstructural Component Checklist	<input type="checkbox"/>	<input checked="" type="checkbox"/> Not Performed at this time

FURTHER EVALUATION REQUIREMENT: Moment Frames and Connections*Material Properties*

To account for uncertainty in the as-built data, a knowledge factor, κ , is determined according to ASCE 41 Table 6-1. Where material properties are listed in the existing construction documents, a knowledge factor of $\kappa=0.9$ is permitted and applied to the component capacities for deformation-controlled and force-controlled actions.

			Default Value per ASCE 41, 4.2.3?	Alternate Value Source?
Concrete			Table (4-2)	Original Book Specification
Slab on Grade	$f'_c=$	2,000 psi	<input type="checkbox"/>	
Cast Flat Slabs, Concrete over Metal Deck, Fireproofing concrete	$f'_c=$	3,000 psi	<input type="checkbox"/>	
Foundation footings and grade beams	$f'_c=$	3,000 psi	<input type="checkbox"/>	
Reinforcing Steel			Table (4-3)	Original Book Specification
All Bars	$f_y=$	60 ksi	<input type="checkbox"/>	
Structural Steel			Tables (4-4), (4-5)	Original Book Specification
Beams & Columns	$F_y=$	36 ksi	<input type="checkbox"/>	
Tubes	$F_y=$	36 ksi	<input type="checkbox"/>	
Welding Electrodes		36 ksi	<input type="checkbox"/>	
Masonry				Original Book Specification
Brick	2,500 psi avg (2,200 psi min) compressive strength			
Mortar	2,000 psi compressive strength			
Grout	2,000 psi compressive strength			

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Buildings 3-5 (1956 Hospital Wing) Summary Data Sheet**BUILDING DATA**

Building Name:	Building 3/4/5 – 1956 Concrete Buildings			Date:	11/12/14
Building Address:	3325 Chanate Road, Santa Rosa, CA 95404				
Latitude:	38.4697	Longitude:	-122.7074	By:	DF
Year Built:	1956	Year(s) Remodeled:	Unknown	Original Design Code:	1952 UBC Assumed
Area (sf):	22k/2.5k/8.4k	Length (ft):	177/81/106	Width (ft):	96/31/105
No. of Stories:	2/1/1	Story Height:	11'-4"	Total Height:	22'-8"/11'-4"/11'-4"

USE ☐ Industrial ☐ Office ☐ Warehouse ☒ Hospital ☐ Residential ☐ Educational ☒ Other: Currently Vacant

CONSTRUCTION DATA

Gravity Load Structural System:	Concrete Bearing Wall and Concrete Beam/Column System		
Exterior Transverse Walls:	Concrete Walls	Openings?	Yes
Exterior Longitudinal Walls:	Concrete Walls	Openings?	Yes
Roof Materials/Framing:	Concrete Slab		
Intermediate Floors/Framing:	Concrete Slab		
Ground Floor:	Slab-on-Grade		
Columns:	Concrete	Foundation:	Spread and Strip
General Condition of Structure:	Good		
Levels Below Grade?	None		
Special Features and Comments:	(4) steel framed bathroom additions on Building 3. Concrete masonry addition (Building 6) on Building 5.		

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Concrete Shear Walls	Concrete Shear Walls
Vertical Elements:	Concrete Shear Walls	Concrete Shear Walls
Diaphragms:	Concrete Slab	Concrete Slab
Connections:	Slab to Wall Dowels	Slab to Wall Dowels

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	$S_{DS} = 1.652 \text{ g}$	$S_{D1} = 0.890 \text{ g}$
Soil Factors:	Class = C	$F_a = 1.0$ $F_v = 1.3$
BSE-1E Spectral Response Accelerations:	$S_{XS} = 0.997 \text{ g}$	$S_{X1} = 0.548 \text{ g}$
Level of Seismicity:	High	Performance Level: Life Safety
Building Period:	$T = 0.208 \text{ sec (2-story)} / 0.124 \text{ sec (1-story)}$	
Spectral Acceleration:	$S_a = 0.997 \text{ g}$	
Modification Factor:	$C_m C_1 C_2 = 1.4$	Building Weight: $W = 3,751/453/1442 \text{ kips}$
Pseudo Lateral Force:	$V = C_m C_1 C_2 S_a W = 1.4 * 0.997 * 3,751 = 5,232 \text{ kips (Bldg 3)}$ $1.4 * 0.997 * 453 = 632 \text{ kips (Bldg 4)}$ $1.4 * 0.997 * 1442 = 2,011 \text{ kips (Bldg 5)}$	

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BUILDING CLASSIFICATION: C2 – Concrete Shear Walls with Stiff Diaphragms**REQUIRED TIER 1 CHECKLISTS**

Yes No

Basic Configuration Checklist

☒ ☐

Building Type C2 Structural Checklist

☒ ☐

Nonstructural Component Checklist

☐ ☒

Not Performed at this time

FURTHER EVALUATION REQUIREMENT: Adjacent Buildings, Surface Fault Rupture, Complete Frames, Coupling Beams**Material Properties**

To account for uncertainty in the as-built data, a knowledge factor, κ , is determined according to ASCE 41 Table 6-1. Where material properties are listed in the existing construction documents, a knowledge factor of $\kappa=0.9$ is permitted and applied to the component capacities for deformation-controlled and force-controlled actions.

			Default Value per ASCE 41, 4.2.3?	Alternate Value Source?
Concrete			Table (4-2)	
Beams:	$f'_c=$	2500 psi	<input type="checkbox"/>	Existing Drawings
Slabs and Columns:	$f'_c=$	2500 psi	<input type="checkbox"/>	Existing Drawings
Walls:	$f'_c=$	2500 psi	<input type="checkbox"/>	Existing Drawings
Reinforcing Steel			Table (4-3)	
#3 Bars:	$f_y=$	40,000 psi	<input type="checkbox"/>	Existing Drawings
#4 Bars and Larger:	$f_y=$	40,000 psi	<input type="checkbox"/>	Existing Drawings

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Building 6 (1956 Hospital Wing) Summary Data Sheet**BUILDING DATA**

Building Name:	Building 6 – 1961 CMU Addition to Building 5	Date:	11/12/14
Building Address:	3325 Chanate Road, Santa Rosa, CA 95404		
Latitude:	38.469455	Longitude:	-122.707509
		By:	BMS
Year Built:	~1961	Year(s) Remodeled:	~1973
Area (sf):	~1,630	Length (ft):	105
No. of Stories:	1	Story Height:	12 ft
		Original Design Code:	1958 UBC Assumed
		Width (ft):	12
		Total Height:	12 ft

USE ☐ Industrial ☐ Office ☐ Warehouse ☒ Hospital ☐ Residential ☐ Educational ☒ Other: Currently Vacant

CONSTRUCTION DATA

Gravity Load Structural System:	5" concrete slab with concrete masonry unit (CMU) and to existing concrete bearing walls		
Exterior Transverse Walls:	N/A (interior walls)	Openings?	N/A
Exterior Longitudinal Walls:	(10) 4'-7" CMU wall piers x 12' tall	Openings?	47% (12) 3.5' windows, 8' entry
Roof Materials/Framing:	5" concrete slab w/ #4@7"oc trans. and #3@18"oc long. over CMU wall w/ steel ledger to 1956 West Wing building concrete wall.		
Intermediate Floors/Framing:	N/A		
Ground Floor:	Elevated (~22" high) 4" slab on grade w/ welded wire mesh. 10' wide stem wall w/ #4@12"oc long.		
Columns:	N/A	Foundation:	14" wide cont ftg w/ (1)#5
General Condition of Structure:	Poor. Critical steel ledger connection is rusted showing significant signs of water damage.		
Levels Below Grade?	None		
Special Features and Comments:	Long narrow addition to the 1956 West Wing building. Telecommunications room added to the west end at a later date of similar concrete masonry unit construction. Critical steel ledger uses archaic expansion anchor system with no reliable tensile capacity.		

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Existing West Wing concrete shear walls on north side. CMU shear walls on south side.	Tied into existing building. (concrete shear walls)
Vertical Elements:	Existing West Wing concrete bearing wall on north side. CMU bearing wall on south side.	N/A
Diaphragms:	Reinforced concrete roof slab.	Reinforced concrete roof slab.
Connections:	Steel ledger with archaic expansion anchorage to existing West Wing concrete wall. Bearing connection with #4@7"oc dowels to CMU walls.	N/A

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	S _{DS} = 1.653 g	S _{D1} = 0.891 g
Soil Factors:	Class= C	F _a = 1.0 F _v = 1.3
BSE-1E Spectral Response Accelerations:	S _{XS} = 0.997g	S _{X1} = 0.548g
Level of Seismicity:	High	Performance Level: Life Safety

3325 Chanate Road, Santa Rosa, CA 95404

Building Period:	T=	0.129 sec	
Spectral Acceleration:	S _a =	0.997g	
Modification Factor:	C _m C ₁ C ₂ =	1.4 (Table 4-8)	Building Weight: W= 223 kips
Pseudo Lateral Force:	V=C _m C ₁ C ₂ S _a W=	1.4*0.997*223 = 310 kips	

BUILDING CLASSIFICATION: RM2 – Reinforced Masonry Walls with Stiff Diaphragms

REQUIRED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Building Type C2 Structural Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Nonstructural Component Checklist	<input type="checkbox"/>	<input checked="" type="checkbox"/> Not Performed at this time

FURTHER EVALUATION REQUIREMENT: Adjacent Buildings, Surface Fault Rupture, Wall Anchorage*Material Properties*

To account for uncertainty in the as-built data, a knowledge factor, κ , is determined according to ASCE 41 Table 6-1. Where material properties are listed in the existing construction documents, a knowledge factor of $\kappa=0.9$ is permitted and applied to the component capacities for deformation-controlled and force-controlled actions.

			Default Value per ASCE 41, 4.2.3?	Alternate Value Source?
<i>Concrete</i>			Table (4-2)	
Slabs:	f _c =	2,500 psi	<input type="checkbox"/>	Existing Drawings
Foundations:	f _c =	2,500 psi	<input type="checkbox"/>	Existing Drawings
<i>Concrete Masonry Units (CMU)</i>			Table (4-3)	
Compressive Strength:	f _c =	1,500 psi	<input type="checkbox"/>	Existing Drawings
<i>Reinforcing Steel</i>			Table (4-3)	
All Bars	f _y =	33,000 psi	<input checked="" type="checkbox"/>	

3325 Chanate Road, Santa Rosa, CA 95404

Building 3 Steel Appendages (1956 Hospital Wing) Summary Data Sheet**BUILDING DATA**

Building Name:	1987 Building 3 Steel Framed Appendages		Date:	12/03/14
Building Address:	3325 Chanate Road, Santa Rosa, CA 95404			
Latitude:	38.4697	Longitude:	-122.7074	By: DF
Year Built:	~1988	Year(s) Remodeled:	N/A	Original Design Code:
Area (sf):	2,100 +/-	Length (ft):	19 (Typical)	Width (ft):
No. of Stories:	2	Story Height:	11'-4"	Total Height:
				22'-8"

USE ☐ Industrial ☐ Office ☐ Warehouse ☒ Hospital ☐ Residential ☐ Educational ☒ Other: Currently Vacant

CONSTRUCTION DATA

Gravity Load Structural System:	Steel Frame		
Exterior Transverse Walls:	Steel Frame	Openings?	N/A
Exterior Longitudinal Walls:	Steel Frame	Openings?	N/A
Roof Materials/Framing:	Metal Decking		
Intermediate Floors/Framing:	Concrete over Metal Decking		
Ground Floor:	Slab-on-Grade		
Columns:	Steel	Foundation:	Spread and Grade Beam
General Condition of Structure:	Good		
Levels Below Grade?	None		
Special Features and Comments:	Directly connected to Building 3. Pre-1994 Code Moment Frame Connections.		

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Steel Moment Frame	Steel Moment Frame
Vertical Elements:	Steel Wide Flange Columns	Steel Wide Flange Columns
Diaphragms:	Concrete over metal decking	Concrete over metal decking
Connections:	Pre-1994 Code Moment Frame Connections	Pre-1994 Code Moment Frame Connections

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	$S_{DS} =$	1.652 g	$S_{D1} =$	0.890 g
Soil Factors:	Class =	C	$F_a =$	1.0
			$F_v =$	1.3
BSE-1E Spectral Response Accelerations:	$S_{XS} =$	0.997g	$S_{X1} =$	0.548g
Level of Seismicity:	High	Performance Level:	Life Safety	
Building Period:	$T =$	0.425 sec		
Spectral Acceleration:	$S_a =$	0.997g		
Modification Factor:	$C_m C_1 C_2 =$	1.1	Building Weight: $W =$	60 kips (Max)
Pseudo Lateral Force:	$V = C_m C_1 C_2 S_a W =$	1.1*0.997*60 = 65.8 kips		

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BUILDING CLASSIFICATION: S1 – Steel Moment Frame w/ Stiff Diaphragms (Floor)
S1A – Steel Moment Frame w/ Flexible Diaphragm (Roof)

REQUIRED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Building Type S1 Structural Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Nonstructural Component Checklist	<input type="checkbox"/>	<input checked="" type="checkbox"/> Not Performed at this time

FURTHER EVALUATION REQUIREMENT: Adjacent Buildings, Surface Fault Rupture, Moment Frames and Connections

Material Properties

To account for uncertainty in the as-built data, a knowledge factor, κ , is determined according to ASCE 41 Table 6-1. Where material properties are listed in the existing construction documents, a knowledge factor of $\kappa=0.75$ is permitted and applied to the component capacities for deformation-controlled and force-controlled actions.

			Default Value per ASCE 41, 4.2.3?	Alternate Value Source?
Concrete			Table (4-2)	
Slab on Grade	$f'_c =$	3,000 psi	<input checked="" type="checkbox"/>	
Cast Flat Slabs, Concrete over Metal Deck, Fireproofing concrete	$f'_c =$	3,000 psi	<input checked="" type="checkbox"/>	
Foundation footings and grade beams	$f'_c =$	3,000 psi	<input checked="" type="checkbox"/>	
Reinforcing Steel			Table (4-3)	
All Bars	$f_y =$	60 ksi	<input checked="" type="checkbox"/>	
Structural Steel			Tables (4-4), (4-5)	
Beams & Columns	$F_y =$	37 ksi	<input checked="" type="checkbox"/>	
Welding Electrodes		36 ksi	<input checked="" type="checkbox"/>	

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Building 7 (1936 Original Hospital Building) Summary Data Sheet**BUILDING DATA**

Building Name:	1936 Hospital Building – Building 7		Date:	11/11/14
Building Address:	3325 Chanate Rd, Santa Rosa, CA 95404			
Latitude:	38.4692	Longitude:	-122.7072	By: JSS
Year Built:	1936	Year(s) Remodeled:	Unknown	Original Design Code:
				1930 UBC Assumed
Area (sf):	37,130	Length (ft):	236	Width (ft): 168.5
No. of Stories:	2 +basement	Story Height:	11'-4"	Total Height: 30'-6"

USE ☐ Industrial ☐ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☒ Other: Unoccupied – To be determined

CONSTRUCTION DATA

Gravity Load Structural System:	Bearing Wall System	
Exterior Transverse Walls:	6" Light Gage Metal Studs @ 16"oc	Openings?
Exterior Longitudinal Walls:	6" Light Gage Metal Studs @ 16"oc	Openings?
Roof Materials/Framing:	Formed 2.5" Concrete over 5:12 Sloped Open Web Steel Joists @ 32"oc	
Intermediate Floors/Framing:	Formed 2.5" Concrete over Open Web Steel Joists @ 32"oc	
Ground Floor:	Formed 2.5" Concrete over Open Web Steel Joists @ 32"oc	
Columns:	Steel Wide Flange at interior	Foundation: Continuous Spread Footings
General Condition of Structure:	Fair	
Levels Below Grade?	Partial Height Basement / Crawlspace	
Special Features and Comments:	No load path for forces into lateral system. Reliance on weak axis bending of light gage.	

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Diagonal Tension Rod Wall Panels	Diagonal Tension Rod Wall Panels
Vertical Elements:	Light Gage Metal Studs	Light Gage Metal Studs
Diaphragms:	2.5" Concrete	2.5" Concrete
Connections:	No Load Path	No Load Path

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	$S_{DS} = 1.651g$	$S_{D1} = 0.890g$
Soil Factors:	Class = C	$F_a = 1.0$ $F_v = 1.3$
BSE-1E Spectral Response Accelerations:	$S_{XS} = 0.997g$	$S_{X1} = 0.549g$
Level of Seismicity:	SDC-E (High)	Performance Level: 3-C (Life Safety)
Building Period:	$T = 0.260$	
Spectral Acceleration:	$S_a = 0.997$	
Modification Factor:	$C_m C_1 C_2 = C = 1.2$ (Tier 1)	Building Weight: $W = 3954$ kips
Pseudo Lateral Force:	$V = C_m C_1 C_2 S_a W = 1.2(0.997)(3954) = 4731$ kips	

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BUILDING CLASSIFICATION: N/A – Does not conform to a standard classification type.

REQUIRED TIER 1 CHECKLISTS

Yes No

Basic Configuration Checklist

☒ ☐

Building Type S2 Structural Checklist

☒ ☐

Building does not conform, but S2 used to check applicable items

Nonstructural Component Checklist

☐ ☒**FURTHER EVALUATION REQUIREMENT:**

By observation, further evaluation beyond Tier 1 will not result in any of the substantially deficient items passing.

Material Properties

To account for uncertainty in the as-built data, a knowledge factor, κ , is determined according to ASCE 41 Table 6-1. Where material properties are listed in the existing construction documents, a knowledge factor of $\kappa=0.9$ is permitted and applied to the component capacities for deformation-controlled and force-controlled actions.

			Default Value per ASCE 41, 4.2.3?	Alternate Value Source?
<i>Concrete</i>			Table (4-2)	
Diaphragm Slabs:	$f'_c=$	2000 psi	<input checked="" type="checkbox"/>	
Basement Walls:	$f'_c=$	2000 psi	<input checked="" type="checkbox"/>	
<i>Reinforcing Steel</i>			Table (4-3)	
#3 Bars:	$f_y=$	33 ksi	<input checked="" type="checkbox"/>	
#4 Bars and Larger:	$f_y=$	33 ksi	<input checked="" type="checkbox"/>	
<i>Structural Steel</i>			Tables (4-4), (4-5)	
Open Web Joists, Wide Flange & Angle sections	$F_y=$	33 ksi	<input checked="" type="checkbox"/>	
Light Gage Metal Studs	$F_y=$	33 ksi	<input checked="" type="checkbox"/>	
Diagonal Tension Rods	$F_y=$	33 ksi	<input checked="" type="checkbox"/>	

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Building 8 (1956 Kitchen/Storage Building) Summary Data Sheet**BUILDING DATA**

Building Name:	Kitchen and Storage – Building 8		Date:	11/11/14
Building Address:	3325 Chanate Road, Santa Rosa, CA			
Latitude:	38.4694	Longitude:	122.7069	By:
Year Built:	1956	Year(s) Remodeled:	Unknown	Original Design Code:
Area (sf):	8600	Length (ft):	210	Width (ft):
No. of Stories:	1	Story Height:	11	Total Height:
				12

USE ☐ Industrial ☐ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☒ Other: Kitchen and storage

CONSTRUCTION DATA

Gravity Load Structural System:	Exterior concrete walls and interior steel beams and columns		
Exterior Transverse Walls:	Cast in place concrete shear walls	Openings?	yes
Exterior Longitudinal Walls:	Cast in place concrete shear walls	Openings?	yes
Roof Materials/Framing:	2x sawn lumber framing with diagonal sheathing		
Intermediate Floors/Framing:	NA		
Ground Floor:	Slab on grade		
Columns:	Exterior cast in place concrete and interior stl pipes	Foundation:	Continuous perimeter ftg
General Condition of Structure:	Good		
Levels Below Grade?	None		
Special Features and Comments:			

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Cast in place concrete shear walls	Cast in place concrete shear walls
Vertical Elements:	Cast in place concrete shear walls	Cast in place concrete shear walls
Diaphragms:	diagonal sheathing	diagonal sheathing
Connections:		

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	S_{DS} =	1.650	S_{D1} =	0.889
Soil Factors:	Class=	C	F_a =	1.002 F_v = 1.41
BSE-1E Spectral Response Accelerations:	S_{XS} =	0.997	S_{X1} =	0.549
Level of Seismicity:	High	Performance Level:	Life Safety	
Building Period:	T=	0.13 seconds		
Spectral Acceleration:	S_a =	1.00		
Modification Factor:	$C_m C_1 C_2$ =	1.1	Building Weight: W=	1,010,000 lbs
Pseudo Lateral Force:	$V=C_m C_1 C_2 S_a W$	1,111,000 lbs		

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BUILDING CLASSIFICATION: C2A – Reinforced concrete shear walls

REQUIRED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Building Type C2A Structural Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Nonstructural Component Checklist	<input type="checkbox"/>	<input checked="" type="checkbox"/>

FURTHER EVALUATION REQUIREMENT:*Material Properties*

To account for uncertainty in the as-built data, a knowledge factor, κ , is determined according to ASCE 41 Table 6-1. Where material properties are listed in the existing construction documents, a knowledge factor of $\kappa=0.9$ is permitted and applied to the component capacities for deformation-controlled and force-controlled actions.

			Default Value per ASCE 41, 4.2.3?	Alternate Value Source?
<i>Concrete</i>			Table (4-2)	
Beams:	$f'_c =$	NA	<input type="checkbox"/>	
Slabs and Columns:	$f'_c =$	2500 psi	<input type="checkbox"/>	Original drawings
Walls:	$f'_c =$	2500 psi	<input type="checkbox"/>	Original drawings
<i>Reinforcing Steel</i>			Table (4-3)	
#3 Bars:	$f_y =$	33 ksi	<input checked="" type="checkbox"/>	
#4 Bars and Larger:	$f_y =$	33 ksi	<input checked="" type="checkbox"/>	
<i>Structural Steel</i>			Tables (4-4), (4-5)	
Beams	$F_y =$	33 ksi	<input checked="" type="checkbox"/>	

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Building 9 (1987 Ambulance Canopy) Summary Data Sheet**BUILDING DATA**

Building Name:	Ambulance Canopy – Building 9		Date:	11/10/2014
Building Address:	3325 Chanate Rd Santa Rosa, CA 95404			
Latitude:	38.469475	Longitude:	-122.707592	By: BMS
Year Built:	~1973	Year(s) Remodeled:	~1988	Original Design Code:
Area (sf):	~625	Length (ft):	30	Width (ft):
No. of Stories:	1	Story Height:	11 ft	Total Height:
				13 ft

USE ☐ Industrial ☐ Office ☐ Warehouse ☒ Hospital ☐ Residential ☐ Educational ☐ Other:

CONSTRUCTION DATA

Gravity Load Structural System:	Verco Type N 20 GA metal deck over steel TS8x6x $\frac{3}{16}$ beams and TS6x6x $\frac{3}{16}$ columns		
Exterior Transverse Walls:	None	Openings?	N/A
Exterior Longitudinal Walls:	None	Openings?	N/A
Roof Materials/Framing:	Verco Type N 20 GA metal deck over steel TS8x6x $\frac{3}{16}$ beams and MC8x8.5 perimeter steel		
Intermediate Floors/Framing:	N/A		
Ground Floor:	Elevated (~22" high) 6" slab on grade with #4 @ 18"oc each way		
Columns:	(4) TS6x6x $\frac{3}{16}$, pinned to slab w/ (4) $\frac{1}{2}$ ø x 3" studs	Foundation:	2' SQ x 12" pad w/ (3)#4 EW
General Condition of Structure:	Acceptable		
Levels Below Grade?	None		
Special Features and Comments:	2" seismic joints at two structures. Rebuilt around 1988 after the damning 1978 Degenkolb report.		

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Pinned cantilevered columns	Pinned cantilevered columns
Vertical Elements:	TS6x6x $\frac{3}{16}$	TS6x6x $\frac{3}{16}$
Diaphragms:	Verco Type N 20 GA metal deck	Verco Type N 20 GA metal deck
Connections:	Welded angle seats at roof and pinned cantilever at the base with 2' SQ pads	Welded angle seats at roof and pinned cantilever at the base with 2' SQ pads

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	S_{DS} =	1.653	S_{D1} =	0.891
Soil Factors:	Class=	C	F_a =	1.0
			F_v =	1.3
BSE-1E Spectral Response Accelerations:	S_{XS} =	0.977	S_{X1} =	0.548
Level of Seismicity:	High	Performance Level:	Life Safety	
Building Period:	T=	0.238		
Spectral Acceleration:	S_a =	0.997		
Modification Factor:	$C_m C_1 C_2$ =	1	Building Weight: W=	8,400 lbs
Pseudo Lateral Force:	$V=C_m C_1 C_2 S_a W$ =	8,375 lbs		

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BUILDING CLASSIFICATION: S1A – Steel Moment Frames with Flexible Diaphragms (Cantilevered Columns)**REQUIRED TIER 1 CHECKLISTS**

	Yes	No
Basic Configuration Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Building Type S1A Structural Checklist	<input checked="" type="checkbox"/>	<input type="checkbox"/>
Nonstructural Component Checklist	<input type="checkbox"/>	<input checked="" type="checkbox"/>

FURTHER EVALUATION REQUIREMENT: Adjacent Buildings, Surface Fault Rupture, Compact Members*Material Properties*

To account for uncertainty in the as-built data, a knowledge factor, κ , is determined according to ASCE 41 Table 6-1. Where material properties are listed in the existing construction documents, a knowledge factor of $\kappa=0.9$ is permitted and applied to the component capacities for deformation-controlled and force-controlled actions.

			Default Value per ASCE 41, 4.2.3?	Alternate Value Source?
<i>Concrete</i>			Table (4-2)	
Slabs and Columns:	$f'_c =$	3 ksi	<input checked="" type="checkbox"/>	
<i>Reinforcing Steel</i>			Table (4-3)	
#3 Bars:	$f_y =$	33 ksi	<input checked="" type="checkbox"/>	
#4 Bars and Larger:	$f_y =$	33 ksi	<input checked="" type="checkbox"/>	
<i>Structural Steel</i>			Tables (4-4), (4-5)	
Columns and Beams	$F_y =$	37 ksi	<input checked="" type="checkbox"/>	

APPENDIX E –TIER 1 CHECKLISTS

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Building 1a - 16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity**Building System****General**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

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Building Configuration

C	NC	N/A	U	
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Medium Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**Geologic Site Hazards**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Foundation Configuration

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

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Building 1a - 16.3LS Building Type W2**Life Safety Structural Checklist For Building Type W2: Wood Frames, Commercial And Industrial**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of W2 building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.3LS Life Safety Structural Checklist for Building Type W2***Building Type W2***

These buildings are commercial or industrial buildings with a floor area of 5,000 square feet or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Seismic forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, oriented strand board, stucco, plaster, straight or diagonal wood sheathing, or braced with rod bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.

Low and Moderate Seismicity**Seismic-Force-Resisting System**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the following values (Commentary: Sec. A.3.2.7.1. Tier 2: Sec. 5.5.3.1.1):
				Structural panel sheathing 1,000 lb/ft
				Diagonal sheathing 700 lb/ft
				Straight sheathing 100 lb/ft
				All other conditions 100 lb/ft
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. (Commentary: Sec. A.3.2.7.2. Tier 2: Sec. 5.5.3.6.1)

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- | | | | | |
|-------------------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard are not used as shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. (Commentary: Sec. A.3.2.7.5. Tier 2: Sec. 5.5.3.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. (Commentary: Sec. A.3.2.7.7. Tier 2: Sec. 5.5.3.6.4) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. (Commentary: Sec. A.3.2.7.8. Tier 2: Sec. 5.5.3.6.5) |

Connections

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WOOD POSTS: There is a positive connection of wood posts to the foundation. (Commentary: Sec. A.5.3.3. Tier 2: Sec. 5.7.3.3) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WOOD SILLS: All wood sills are bolted to the foundation. (Commentary: Sec. A.5.3.4. Tier 2: Sec. 5.7.3.3) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | GIRDER/COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1) |

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity**Diaphragms**

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1) |

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- | | | | | |
|-------------------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. (Commentary: Sec. A.4.1.3. Tier 2: Sec. 5.6.1.1) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5) |

Connections

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WOOD SILL BOLTS: Sill bolts are spaced at 6 ft or less with proper edge and end distance provided for wood and concrete. (Commentary: Sec. A.5.3.7. Tier 2: Sec. 5.7.3.3) |

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Building 1b - 16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity**Building System****General**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

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Building Configuration

C	NC	N/A	U	
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Medium Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**Geologic Site Hazards**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Foundation Configuration

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

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Building 1b - 16.3LS Building Type W2**Life Safety Structural Checklist For Building Type W2: Wood Frames, Commercial And Industrial**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of W2 building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.3LS Life Safety Structural Checklist for Building Type W2***Building Type W2***

These buildings are commercial or industrial buildings with a floor area of 5,000 square feet or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Seismic forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, oriented strand board, stucco, plaster, straight or diagonal wood sheathing, or braced with rod bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.

Low and Moderate Seismicity**Seismic-Force-Resisting System**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the following values (Commentary: Sec. A.3.2.7.1. Tier 2: Sec. 5.5.3.1.1):
				Structural panel sheathing 1,000 lb/ft
				Diagonal sheathing 700 lb/ft
				Straight sheathing 100 lb/ft
				All other conditions 100 lb/ft
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. (Commentary: Sec. A.3.2.7.2. Tier 2: Sec. 5.5.3.6.1)

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|-------------------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard are not used as shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. (Commentary: Sec. A.3.2.7.5. Tier 2: Sec. 5.5.3.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. (Commentary: Sec. A.3.2.7.7. Tier 2: Sec. 5.5.3.6.4) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. (Commentary: Sec. A.3.2.7.8. Tier 2: Sec. 5.5.3.6.5) |

Connections

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|-------------------------------------|--------------------------|-------------------------------------|--------------------------|---|
| C | NC | N/A | U | |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | WOOD POSTS: There is a positive connection of wood posts to the foundation. (Commentary: Sec. A.5.3.3. Tier 2: Sec. 5.7.3.3) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WOOD SILLS: All wood sills are bolted to the foundation. (Commentary: Sec. A.5.3.4. Tier 2: Sec. 5.7.3.3) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | GIRDER/COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1) |

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity**Diaphragms**

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|-------------------------------------|--------------------------|--------------------------|--------------------------|--|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1) |

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|-------------------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. (Commentary: Sec. A.4.1.3. Tier 2: Sec. 5.6.1.1) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5) |

Connections

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WOOD SILL BOLTS: Sill bolts are spaced at 6 ft or less with proper edge and end distance provided for wood and concrete. (Commentary: Sec. A.5.3.7. Tier 2: Sec. 5.7.3.3) |

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Building 1c - 16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity**Building System****General**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

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Building Configuration

C	NC	N/A	U	
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Medium Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**Geologic Site Hazards**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Foundation Configuration

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

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Building 1c - 16.3LS Building Type W2**Life Safety Structural Checklist For Building Type W2: Wood Frames, Commercial And Industrial**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of W2 building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.3LS Life Safety Structural Checklist for Building Type W2***Building Type W2***

These buildings are commercial or industrial buildings with a floor area of 5,000 square feet or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Seismic forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, oriented strand board, stucco, plaster, straight or diagonal wood sheathing, or braced with rod bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.

Low and Moderate Seismicity**Seismic-Force-Resisting System**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the following values (Commentary: Sec. A.3.2.7.1. Tier 2: Sec. 5.5.3.1.1): Structural panel sheathing 1,000 lb/ft Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. (Commentary: Sec. A.3.2.7.2. Tier 2: Sec. 5.5.3.6.1)

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|-------------------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard are not used as shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. (Commentary: Sec. A.3.2.7.5. Tier 2: Sec. 5.5.3.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. (Commentary: Sec. A.3.2.7.7. Tier 2: Sec. 5.5.3.6.4) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. (Commentary: Sec. A.3.2.7.8. Tier 2: Sec. 5.5.3.6.5) |

Connections

- | | | | | |
|-------------------------------------|--------------------------|-------------------------------------|--------------------------|---|
| C | NC | N/A | U | |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | WOOD POSTS: There is a positive connection of wood posts to the foundation. (Commentary: Sec. A.5.3.3. Tier 2: Sec. 5.7.3.3) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WOOD SILLS: All wood sills are bolted to the foundation. (Commentary: Sec. A.5.3.4. Tier 2: Sec. 5.7.3.3) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | GIRDER/COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1) |

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity**Diaphragms**

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1) |

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- | | | | | |
|-------------------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. (Commentary: Sec. A.4.1.3. Tier 2: Sec. 5.6.1.1) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5) |

Connections

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WOOD SILL BOLTS: Sill bolts are spaced at 6 ft or less with proper edge and end distance provided for wood and concrete. (Commentary: Sec. A.5.3.7. Tier 2: Sec. 5.7.3.3) |

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Building 2 - 16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity**Building System****General**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) <i>Further Tier 2 / Tier 3 analysis is required</i>
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

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Building Configuration

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Medium Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**Geologic Site Hazards**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

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☐ ☒ ☐ ☐ SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

Per Rutherford & Chekene Geotech Report #2002-112G, dated 12/20/2002

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Foundation Configuration

C NC N/A U

☒ ☐ ☐ ☐ OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

☒ ☐ ☐ ☐ TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

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Building 2 - 16.4LS Building Type S1 and S1A**Life Safety Structural Checklist For Building Type S1: Steel Moment Frames With Stiff Diaphragms And Type S1A: Steel Moment Frames With Flexible Diaphragms**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of S1 or S1A building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.4LS Life Safety Structural Checklist for Building Type S1 and S1A***Building Type S1***

These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists, or steel trusses. Seismic forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames.

Where the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. Foundations consist of concrete spread footings or deep pile foundations.

Refer to Section A.3.1.3 for a general discussion of steel moment frames.

Building Type S1A

These buildings are similar to Building Type S1, except that the diaphragms consist of wood framing; untopped metal deck; or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping and are flexible relative to the frames.

Low Seismicity**Seismic-Force-Resisting System**

C NC N/A U

☒ ☐ ☐ ☐

DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.5.3.1, is less than 0.025. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2)

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- ☒ ☐ ☐ ☐ COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)
- ☐ ☒ ☐ ☐ FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9, is less than F_y . Columns need not be checked if the strong column–weak beam checklist item is compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2)

*Further Tier 2 / Tier 3 analysis is required***Connections**

C NC N/A U

- ☒ ☐ ☐ ☐ TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)

- ☒ ☐ ☐ ☐ STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**Seismic-Force-Resisting-System**

C NC N/A U

- ☒ ☐ ☐ ☐ REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1)
- ☒ ☐ ☐ ☐ INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1)
- ☐ ☒ ☐ ☐ MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). Note: more restrictive requirements for High Seismicity.

Further Tier 2 / Tier 3 analysis is required

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High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Seismic-Force-Resisting-System

C	NC	N/A	U	
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel per AISC 341, Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1)
				<i>Further Tier 2 / Tier 3 analysis is required</i>
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. (Commentary: Sec. A.3.1.3.5. Tier 2: Sec. 5.5.2.2.2)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3)
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	STRONG COLUMN—WEAK BEAM: The percentage of strong column—weak beam joints in each story of each line of moment frames is greater than 50%. (Commentary: Sec. A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5)
				<i>Further Tier 2 / Tier 3 analysis is required</i>
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	COMPACT MEMBERS: All frame elements meet section requirements set forth by AISC 341 Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4)

Diaphragms (Stiff or Flexible)

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)

Flexible Diaphragms

C	NC	N/A	U	
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

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- | | | | | |
|-------------------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5) |

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Buildings 3-6 - 16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity**Building System****General**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) <i>Existing 8" seismic joint at adjacent 2-story 1936 and 1972 Acute Care Buildings. Minimum seismic joint for 22'-8" height is 10.9". (See Calculations). Existing seismic joint between Buildings 3 and 6 is insufficient (See Calculations). No seismic joint between Building 3 and steel framed appendages.</i>
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

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Building Configuration

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)
				(See Calculations).

Medium Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**Geologic Site Hazards**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

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☐ ☒ ☐ ☐ SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Foundation Configuration

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

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Buildings 3-5 - 16.10LS Building Type C2 and C2A Checklist**Life Safety Structural Checklist For Building Type C2: Concrete Shear Walls With Stiff Diaphragms, Type C2A: Concrete Shear Walls With Flexible Diaphragms**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of C2 or C2A building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16. 10LS Life Safety Structural Checklist for Building Type C2 and C2A***Building Type C2***

These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Floors are supported on steel beams and columns or on concrete beams and columns or bearing walls. Seismic forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced but often extend throughout the building. In more recent construction, shear walls occur in isolated locations and are more heavily reinforced with boundary elements and closely spaced ties to provide ductile performance. The diaphragms consist of concrete slabs and are stiff relative to the walls. Foundations consist of concrete spread footings, mat foundations, or deep foundations.

Building Type C2A

These buildings are similar to C2 except that the diaphragms consist of wood sheathing, untopped metal deck; or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping or have large aspect ratios, and are flexible relative to the walls.

Refer to Sections A.3.2.1 and A3.2.2 for additional commentary related to concrete shear walls.

Low and Moderate Seismicity**Seismic-Force-Resisting System**

C NC N/A U

☐ ☒ ☐ ☐ COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5.2.5.1)

(See Calculations).

☒ ☐ ☐ ☐ REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)

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- ☒ ☐ ☐ ☐ SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 lb/in.^2 or $2\sqrt{f_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)

(See Calculations).

- ☒ ☐ ☐ ☐ REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)

(See Calculations).

Connections

C NC N/A U

- ☐ ☐ ☒ ☐ WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)

- ☒ ☐ ☐ ☐ TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)

- ☒ ☐ ☐ ☐ FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Seismic-Force-Resisting System

C NC N/A U

- ☒ ☐ ☐ ☐ DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)

(See Calculations).

- ☒ ☐ ☐ ☐ FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)

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- ☐ ☒ ☐ ☐ COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than $d/2$ and are anchored into the confined core of the beam with hooks of 135 degrees or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)

(Coupling beam stirrups do not have 135 degree hooks).

Connections

C NC N/A U

- ☐ ☐ ☒ ☐ UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)

Diaphragms (Stiff or Flexible)

C NC N/A U

- ☒ ☐ ☐ ☐ DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)
- ☒ ☐ ☐ ☐ OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)

Flexible Diaphragms

C NC N/A U

- ☐ ☐ ☒ ☐ CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)
- ☐ ☐ ☒ ☐ STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)
- ☐ ☐ ☒ ☐ SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)
- ☐ ☐ ☒ ☐ DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)
- ☐ ☐ ☒ ☐ OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

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Building 6 - 16.15LS Building Type RM1 and RM2**Life Safety Structural Checklist For Building Types RM1: Reinforced Masonry Bearing Walls With Flexible Diaphragms And RM2: Reinforced Masonry Bearing Walls With Stiff Diaphragms**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of RM1 or RM2 building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.15LS Life Safety Structural Checklist for Building Type RM1 and RM2***Building Type RM1***

These buildings have bearing walls that consist of reinforced brick or concrete block masonry. Wood floor and roof framing consists of wood joists, glulam beams, and wood posts or small steel columns. Steel floor and roof framing consists of steel beams or open web joists, steel girders, and steel columns. Seismic forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, untopped metal deck, or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping and are flexible relative to the walls. Foundations consist of brick or concrete spread footings or deep foundations.

Building Type RM2

These buildings similar to Building Type RM except that diaphragms consist of metal deck with concrete fill, precast concrete planks, tees, or double-tees, with or without a cast-in-place concrete topping slab, and are stiff relative to the walls.

Low and Moderate Seismicity**Seismic-Force-Resisting System**

C NC N/A U

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 lb/in. ² . (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1) |

See calculations. Shear walls piers are likely flexural governed due to their 3:1 height-to-width ratio.

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- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3) |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|

See calculations

Stiff Diaphragms

C NC N/A U

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|

Roof slab is cast in place.

Connections

C NC N/A U

- | | | | | |
|--------------------------|-------------------------------------|--------------------------|--------------------------|---|
| <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1) |
|--------------------------|-------------------------------------|--------------------------|--------------------------|---|

Exterior CMU walls are doweled into the roof slab but the slab is connected to the existing building using an archaic expansion anchor with little to no tensile capacity.

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|

- | | | | | |
|--------------------------|-------------------------------------|--------------------------|--------------------------|---|
| <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2) |
|--------------------------|-------------------------------------|--------------------------|--------------------------|---|

Steel ledger with 7/8" MB @ 3'-0"oc and archaic expansion anchors at existing building. Steel ledger is visibly deteriorated with rust from roof leak. #4 @ 7"oc dowels plus shear friction at new CMU walls.

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|

Roof slab appears to be cast in place.

- | | | | | |
|--------------------------|--------------------------|--------------------------|-------------------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4) |
|--------------------------|--------------------------|--------------------------|-------------------------------------|--|

#4 dowels to match spacing of wall reinforcement but wall reinforcement is (4)#5 at wall boundaries. Is likely compliant but should be verified in field that dowels match the size and number of boundary reinforcement to provide fixed base shear wall pier condition.

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- ☐ ☐ ☒ ☐ GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Stiff Diaphragms

- | C | NC | N/A | U | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <p>OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)</p> <p><i>No slab openings.</i></p> |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <p>OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)</p> <p><i>No slab openings.</i></p> |

Flexible Diaphragms

- | C | NC | N/A | U | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <p>CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)</p> <p><i>Concrete roof slab.</i></p> |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <p>OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)</p> <p><i>Concrete roof slab.</i></p> |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <p>OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)</p> <p><i>Concrete roof slab.</i></p> |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <p>STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)</p> <p><i>Concrete roof slab.</i></p> |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <p>SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)</p> <p><i>Concrete roof slab.</i></p> |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <p>DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)</p> <p><i>Concrete roof slab.</i></p> |

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<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)
--------------------------	--------------------------	-------------------------------------	--------------------------	--

Concrete roof slab.

Connections

C	NC	N/A	U
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<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)
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Building 3 Appendages - 16.4LS Building Type S1 and S1A Checklist**Life Safety Structural Checklist For Building Type S1: Steel Moment Frames With Stiff Diaphragms And Type S1A: Steel Moment Frames With Flexible Diaphragms**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of S1 or S1A building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.4LS Life Safety Structural Checklist for Building Type S1 and S1A***Building Type S1***

These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists, or steel trusses. Seismic forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames.

Where the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. Foundations consist of concrete spread footings or deep pile foundations.

Refer to Section A.3.1.3 for a general discussion of steel moment frames.

Building Type S1A

These buildings are similar to Building Type S1, except that the diaphragms consist of wood framing; untopped metal deck; or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping and are flexible relative to the frames.

Low Seismicity**Seismic-Force-Resisting System**

C NC N/A U

☐ ☒ ☐ ☐

DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.5.3.1, is less than 0.025. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2)

(See Calculations).

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- ☒ ☐ ☐ ☐ COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)
- (See Calculations).
- ☐ ☒ ☐ ☐ FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9, is less than F_y . Columns need not be checked if the strong column–weak beam checklist item is compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2)
- (See Calculations).

Connections

C NC N/A U

- ☒ ☐ ☐ ☐ TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)
- ☒ ☐ ☐ ☐ STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity

Seismic-Force-Resisting-System

C NC N/A U

- ☒ ☐ ☐ ☐ REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1)
- ☐ ☐ ☒ ☐ INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1)
- ☐ ☒ ☐ ☐ MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). Note: more restrictive requirements for High Seismicity.

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High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Seismic-Force-Resisting-System

C	NC	N/A	U	
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<p>MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel per AISC 341, Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1)</p> <p>(See Calculations).</p>
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<p>PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. (Commentary: Sec. A.3.1.3.5. Tier 2: Sec. 5.5.2.2.2)</p> <p>(See Calculations).</p>
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<p>COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3)</p>
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<p>STRONG COLUMN—WEAK BEAM: The percentage of strong column—weak beam joints in each story of each line of moment frames is greater than 50%. (Commentary: Sec. A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5)</p> <p>(See Calculations).</p>
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<p>COMPACT MEMBERS: All frame elements meet section requirements set forth by AISC 341 Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4)</p> <p>(See Calculations).</p>

Diaphragms (Stiff or Flexible)

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<p>OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)</p>

Flexible Diaphragms

C	NC	N/A	U	
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<p>CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)</p>
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<p>STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)</p>

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- | | | | | |
|-------------------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5) |

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Building 7 - 16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity**Building System****General**

C	NC	N/A	U	
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1) <i>No shear transfer from roof to walls, no shear transfer from floors to walls, minimal collectors/chords along wall lines at roof/floors, no collectors at re-entrant corners across wing projections.</i>
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) <i>1956 Addition seismic gap = 8" < 4% x 24' = 11.5" (NC)</i> <i>1956 Storage/Kitchen seismic gap = 12" (C)</i>

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|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|

Building Configuration

- | | | | | |
|-------------------------------------|-------------------------------------|--------------------------|--------------------------|--|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5) |
| <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) |

Discontinuous diaphragms at full height wall studs. Independent diaphragms have large torsional eccentricity by observation.

Medium Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**Geologic Site Hazards**

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|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) |

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<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)
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<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)
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Estimated fault locations runs through the building.

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Foundation Configuration

C NC N/A U

<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)
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<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)
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Building 7 - 16.5LS Building Type S2 and S2A

Life Safety Structural Checklist For Building Types S2: Steel Braced Frames With Stiff Diaphragms And S2A: Steel Braced Frames With Flexible Diaphragms

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of S2 or S2A building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.5LS Life Safety Structural Checklist for Building Type S2 and S2A

Building Type S2

These buildings have a frame of steel columns, beams, and braces. Braced frames develop resistance to seismic forces by the bracing action of the diagonal members. The braces induce forces in the associated beams and columns such that all elements work together in a manner similar to a truss with all element stresses being primarily axial. Where the braces do not completely triangulate the panel, some of the members are subjected to shear and flexural stresses; eccentrically braced frames are one such case.

The diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames.

Where the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural furring. Foundations consist of concrete spread footings or deep pile foundations.

Refer to Section A.3.3 for additional general commentary on braced frames, Section A.3.3.2 for concentrically braced frames, and Section A.3.3.3 for eccentrically braced frames.

Building Type S2A

These buildings are similar to Building Type S2A except that diaphragms consist of wood framing; untopped metal deck; or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping and are flexible relative to the frames.

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Low Seismicity

Seismic-Force-Resisting System

C NC N/A U

- ☐ ☒ ☐ ☐ COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)

Wall panel edge stud demand to capacity ratios vary from 8.4 to 9.5, substantially deficient, using the $0.30F_y$ criteria.

- ☐ ☒ ☐ ☐ BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.5.3.4, is less than $0.50F_y$. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1)

Diagonal rod brace demand to capacity ratios vary from 7.5 to 10.5, substantially deficient.

Connections

C NC N/A U

- ☐ ☒ ☐ ☐ TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)

Diaphragms are not directly connected thru the floor joist truss heels to the ledgers. Ledgers are not directly connected to the diagonal rods or to the top tracks or sill tracks. Reliance upon weak-axis bending of the truss heels and weak-axis bending of thin gage studs.

- ☐ ☒ ☐ ☐ STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)

Wall panel edge studs are not directly connected to the foundation. Reliance upon weak-axis bending of the sill tracks and welds from stud to track.

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity

Seismic-Force-Resisting-System

C NC N/A U

- ☒ ☐ ☐ ☐ REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2. The number of braced bays in each line is greater than 2. (Commentary: Sec. A.3.3.1.1. Tier 2: Sec. 5.5.1.1)

- ☐ ☐ ☒ ☐ CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4)

Tension only rods, not applicable.

- ☐ ☒ ☐ ☐ COMPACT MEMBERS: All brace elements meet compact section requirements set forth by AISC 360, Table B4.1. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)

Steel stud "columns" not compact.

- ☒ ☐ ☐ ☐ K-BRACING: The bracing system does not include K-braced bays. (Commentary: Sec. A.3.3.2.1. Tier 2: Sec. 5.5.4.6)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Seismic-Force-Resisting-System

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	COLUMN SPLICES: All column splice details located in braced frames develop 50% of the tensile strength of the column. (Commentary: Sec. A.3.3.1.3. Tier 2: Sec. 5.5.4.2) <i>Wall panel edge studs continuous.</i>
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	SLENDERNES OF DIAGONALS: All diagonal elements required to carry compression have Kl / r ratios less than 200. (Commentary: Sec. A.3.3.1.4. Tier 2: Sec. 5.5.4.3) <i>Tension only rods, not applicable.</i>
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4) <i>By observation, connections do <u>not</u> develop rods.</i>
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	COMPACT MEMBERS: All brace elements meet section requirements set forth by AISC 341, Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4) <i>Wall panel edge studs not compact.</i>
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs. (Commentary: Sec. A.3.3.2.3. Tier 2: Sec. 5.5.4.6)
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces shall frame into the beam–column joints concentrically. (Commentary: Sec. A.3.3.2.4. Tier 2: Sec. 5.5.4.8) <i>Lateral eccentricities from center of rods to ledger angle collectors at face of studs.</i>

Diaphragms (Stiff or Flexible)

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25% of the frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)

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Building 8 - 16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity**Building System****General**

C NC N/A U

☒ ☐ ☐ ☐ **LOAD PATH:** The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

☐ ☒ ☐ ☐ **ADJACENT BUILDINGS:** The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

There is a 12" seismic separation between the building and adjacent 1936 wing which is compliant. At the north end of the structure the gap to the adjacent building is less than 4% however the building elements are offset vertically at the joint to prevent interaction and therefore are compliant.

The covered walkway at the north end of the structure is tied to the building and has only a nominal separation from the adjacent 1936 building, in that location the structure is non-compliant, for both the tier 1 and Tier 2 checks.

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|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|

Building Configuration

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| C | NC | N/A | U | |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) |

Medium Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**Geologic Site Hazards**

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|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| C | NC | N/A | U | |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) |

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☒ ☐ ☐ ☐ SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

☒ ☐ ☐ ☐ SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)
The building is within the Alquist-Priolo zone for the Healdsburg-Rodgers Creek Fault based on 1983 mapping. However subsequent site specific fault trace mapping has not located a fault trace at this building. Several fault traces have been located on the site, generally the identified fault traces are to the north and west of this structure.

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Foundation Configuration

C NC N/A U

☒ ☐ ☐ ☐ OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

☐ ☐ ☒ ☐ TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

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Building 8 - 16.10LS Building Type C2 and C2A**Life Safety Structural Checklist For Building Type C2: Concrete Shear Walls With Stiff Diaphragms, Type C2A: Concrete Shear Walls With Flexible Diaphragms**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of C2 or C2A building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16. 10LS Life Safety Structural Checklist for Building Type C2 and C2A***Building Type C2***

These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Floors are supported on steel beams and columns or on concrete beams and columns or bearing walls. Seismic forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced but often extend throughout the building. In more recent construction, shear walls occur in isolated locations and are more heavily reinforced with boundary elements and closely spaced ties to provide ductile performance. The diaphragms consist of concrete slabs and are stiff relative to the walls. Foundations consist of concrete spread footings, mat foundations, or deep foundations.

Building Type C2A

These buildings are similar to C2 except that the diaphragms consist of wood sheathing, untopped metal deck; or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping or have large aspect ratios, and are flexible relative to the walls.

Refer to Sections A.3.2.1 and A3.2.2 for additional commentary related to concrete shear walls.

Low and Moderate Seismicity**Seismic-Force-Resisting System**

C NC N/A U

☐ ☐ ☒ ☐

COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5.2.5.1)

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|-------------------------------------|--------------------------|--------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 lb/in.^2 or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3) |

Connections

C NC N/A U

- | | | | | |
|-------------------------------------|-------------------------------------|--------------------------|--------------------------|--|
| <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <p>WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)</p> <p><i>The wall anchorage in the east-west direction (perpendicular to framing) is not adequate per the Tier 1 and 2 analysis, the connection capacity is limited by the current number of nails to the wood framing. The capacity of the anchor itself and the connection to the concrete is adequate per the Tier 1 quick check procedures.</i></p> <p><i>The wall anchorage in the north-south direction is adequate per the Tier 1 and Tier 2 analysis.</i></p> |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4) |

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Seismic-Force-Resisting System

C NC N/A U

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|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3) |

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|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than $d/2$ and are anchored into the confined core of the beam with hooks of 135 degrees or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|

Connections

C NC N/A U

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|

Diaphragms (Stiff or Flexible)

C NC N/A U

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3) |

Flexible Diaphragms

C NC N/A U

- | | | | | |
|-------------------------------------|-------------------------------------|-------------------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) |
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) |
| <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) |

The roof diaphragm is diagonal sheathing which spans greater than 40 feet in each direction with a worst case span of 110 feet. Aspect ratios are less than 4-to-1 and are compliant. The diaphragms are not adequate per the Tier 2 analysis.

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- ☒ ☐ ☐ ☐ OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

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Building 9 - 16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity**Building System****General**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1) <i>Pinned cantilevered Tube Steel (TS) columns welded to TS roof framing with steel angles.</i>
<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) <i>0.04*(11*12")=5.28" Seismic gap is 2 inches to structure but less than one inch to finishes (flashing and gutters) in some locations.</i>
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

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Building Configuration

C	NC	N/A	U	
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

*Flexible Diaphragm***Medium Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity****Geologic Site Hazards**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

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☐ ☒ ☐ ☐ SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity

Foundation Configuration

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

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Building 9 - 16.4LS Building Type S1 and S1A**Life Safety Structural Checklist For Building Type S1: Steel Moment Frames With Stiff Diaphragms And Type S1A: Steel Moment Frames With Flexible Diaphragms**

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of S1 or S1A building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.4LS Life Safety Structural Checklist for Building Type S1 and S1A**Building Type S1**

These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists, or steel trusses. Seismic forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames.

Where the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. Foundations consist of concrete spread footings or deep pile foundations.

Refer to Section A.3.1.3 for a general discussion of steel moment frames.

Building Type S1A

These buildings are similar to Building Type S1, except that the diaphragms consist of wood framing; untopped metal deck; or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping and are flexible relative to the frames.

Low Seismicity**Seismic-Force-Resisting System**

C	NC	N/A	U	
<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.5.3.1, is less than 0.025. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2)
<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)

Cantilevered columns are not subjected to overturning

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- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9, is less than F_y . Columns need not be checked if the strong column–weak beam checklist item is compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2) |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|

Connections

C NC N/A U

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2) |
|-------------------------------------|--------------------------|--------------------------|--------------------------|---|

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1) |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|

*Welded to 1/2" base plate with (4) 3/4" ϕ AB x12" and encased in a reinforced 18" SQ concrete pilaster***Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity**

Seismic-Force-Resisting-System

C NC N/A U

- | | | | | |
|---|--------------------------|-------------------------------------|--------------------------|---|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) |
| <i>4 cantilevered columns total, 2 per drag line. Number of bays is not applicable.</i> | | | | |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1) |
| <i>No walls around the canopy.</i> | | | | |
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). Note: more restrictive requirements for High Seismicity. |

*Cantilevered Columns***High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity**

Seismic-Force-Resisting-System

C NC N/A U

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel per AISC 341, Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|

Cantilevered Columns

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- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. (Commentary: Sec. A.3.1.3.5. Tier 2: Sec. 5.5.2.2.2) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|

Cantilevered Columns

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|

One Story

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | STRONG COLUMN—WEAK BEAM: The percentage of strong column—weak beam joints in each story of each line of moment frames is greater than 50%. (Commentary: Sec. A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|

Cantilevered Columns

- | | | | | |
|--------------------------|-------------------------------------|--------------------------|--------------------------|--|
| <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | COMPACT MEMBERS: All frame elements meet section requirements set forth by AISC 341 Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4) |
|--------------------------|-------------------------------------|--------------------------|--------------------------|--|

*TS6x6x 3/16 36 ksi does not meet Moderately Ductile requirements per AISC 341 Table D1.1***Diaphragms (Stiff or Flexible)**

C NC N/A U

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|---|

*Cantilevered columns have no length.***Flexible Diaphragms**

C NC N/A U

- | | | | | |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|
| <input checked="" type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) |
|-------------------------------------|--------------------------|--------------------------|--------------------------|--|

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|

Metal Deck

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|

Metal Deck

- | | | | | |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|
| <input type="checkbox"/> | <input type="checkbox"/> | <input checked="" type="checkbox"/> | <input type="checkbox"/> | DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) |
|--------------------------|--------------------------|-------------------------------------|--------------------------|--|

Metal Deck

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☒ ☐ ☐ ☐

OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

Metal Deck

APPENDIX F – STRUCTURAL CALCULATIONS

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Building 1 (1999-2004 Cath Lab)1999 — MODULAR CATH LABS:

4916 ft² — 1998 CBC (1997 UBC > 1976 UBC)
 TOTAL SQ. FT. ∴ BENCHMARK BLDG.
 RDL = 18.5 psf. EXT. WALL = 15 psf.

$$W = (18.5)(4916) + (15 \text{ psf})(306')\left(\frac{1}{2} + 2'\right) + (8)(4916) = 167,000^{\#}$$

↑ INT. PARTITION.

$$\text{MECH. UNITS} = 200 + 110 + 100 + (3)(800) + (8 \text{ psf})(4.5)(192') = 9722^{\#}$$

↑ MECH. SEEN.

$$W_T = 167,000 + 9722 = 177,000^{\#}$$

$$S_a = 0.996; C = 1.3$$

$$V_j = (1.3)(0.996)(177,000) = 229,180 \text{ lbs}$$

$$A_w = L_{w_T} = 27.5 + 18.7 + 31.9 = 78.1' \text{ TRANSVERSE DIR.}$$

$$A_w = L_{w_L} = 9.67' + 13.5 + 30.7 + 36.1 + 49.5 = 139.5' \text{ LONGITUDINAL DIR.}$$

TRANS. CONTROLS.

$$N_j^{avg} = \left(\frac{1}{4}\right)\left(\frac{229,180}{78.1}\right) = 733.6 \text{ PLF} < 1000 \text{ PLF}$$

∴ COMPLIES.

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2001 MRI ADDITION:

$$S_A = 0.996; C = 1.3$$

$$V_j = (1.3)(0.996)(37,730) = 48,853$$

$$A_w = 23' + 17' = 40'$$

↑ MOST CRITICAL IN
TRANSVERSE DIRECTION

$$N_j^{AVE} = \left(\frac{1}{4}\right)\left(\frac{48,853}{40'}\right) = 305 \text{ PLF} < 1000 \text{ PLF}$$

COMPLIES

→ SEE CALC FOR ROOF ADDITION.

1998 CBC: (1997 UBC) BENCHMARK
AREA = 1138 ft²

$$\text{Roof Dead} = 15.5 \text{ psf.}$$

$$\text{EXT WALL DL} = 15 \text{ psf.}$$

$$W = (15.5)(1138) + (15)\left(\frac{12}{2} + 2\right)(120') = 32,040^{\#}$$

$$+ (5)(1138) = 37,730^{\#}$$

↑ INT. PART.

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2004 CATH LAB ADD:

$$V = C S_a W :$$

$$T = (0.020)^{0.75} (14') = 0.145s$$

$$S_a = \frac{0.548}{0.145} = 3.78$$

$$\text{But } S_a \leq S_{xs} = 0.996$$

$$S_a = 0.996$$

$$C = 1.3$$

$$V = (1.3)(.996)W = 1.30W$$

DETERMINE W:

$$\text{TOTAL SQ FT} = 1262$$

$$RDL = 18.5 \text{ psf}$$

$$W = (18.5 + 11 + 5)(1262) + 6070^{\#}$$

↑
ROOF UNITS

$$\text{EXTERIOR WALLS} = \frac{(15 \text{ psf})(12' + 12')(1705)}{1262 \text{ ft}^2}$$

$$= 11 \text{ psf}$$

$$\rightarrow W = 70,309^{\#}$$

$$\text{INT. WALLS} = 5 \text{ psf}$$

$$V_j = (1.30)(70,309) = 91,402^{\#}$$

SHEAR
STRESS
CHECK,

$$A_w = 17' + 30' = 47'$$

(MOST CRITICAL
IN TRANSVERSE)

$$M_s = 4.0$$

$$V_j^{\text{AVG}} = \left(\frac{1}{4}\right) \left(\frac{91,402}{47'}\right) = 486 \text{ PLF} < 1000 \text{ PLF}$$

∴ COMPLIES.

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Building 2 (1972 Acute Care Hospital)

Adjacent Buildings (Sec A2.1.2)				
h1	24	ft		
h2	12.5	ft		
Required				
Separation1	11.52	inches		
Separation2	6	inches		
Actual				
Separation1	4	inches	NOT COMPLIANT	
Separation2	4	inches	NOT COMPLIANT	

Overtuning (Tier 1 Sec A.6.2.1)				
Sa	0.58			
0.6 Sa	0.348			
b/h (e-w)	2.35102	>	0.348	COMPLIANT
b/h (n-s)	1.469388	>	0.348	COMPLIANT
The foundation is adequate for overturning.				

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Seismic Story Force Distribution based on ASCE 41-13							
				Ta Period (7.4.1.2.2)= 0.942		k= 1.221	(7.4.1.3.2)

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Penthouse weight combined to main roof**Seismic Story Force Distribution based on ASCE 41-13**

			Ta Period (7.4.1.2.2)= 0.942		k= 1.221	(7.4.1.3.2)
--	--	--	------------------------------	--	----------	-------------

V(ULT)= 0.582	Base V (ULT)= 3,103,260
---------------	-------------------------

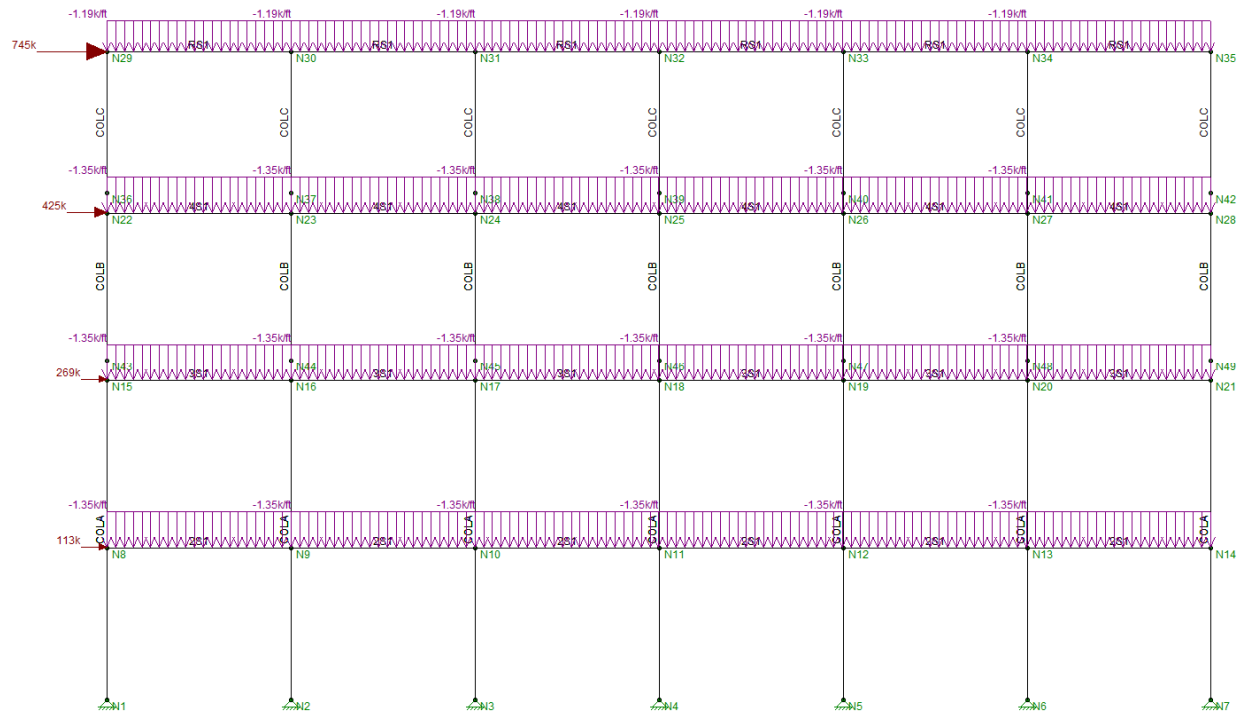
Story Force Vertical Distribution (ASCE 41-13 7.4.1.3.2)

Level	w_x	h_x (ft.)	h_x^k	$w_x h_x^k$	F_x , ULT	Load to frame	$Cv_x\%$
ROOF	1,484,763	48.8	115.3	171215716	1,489,181	744,591	48.0
4th	1,215,413	36.3	80.4	97680512	849,595	424,797	27.4
3rd	1,288,088	23.8	48.0	61860580	538,044	269,022	17.3
2nd	1,343,798	11.3	19.4	26034468	226,440	113,220	7.3
Σ	5,332,062			356791277	3,103,260		

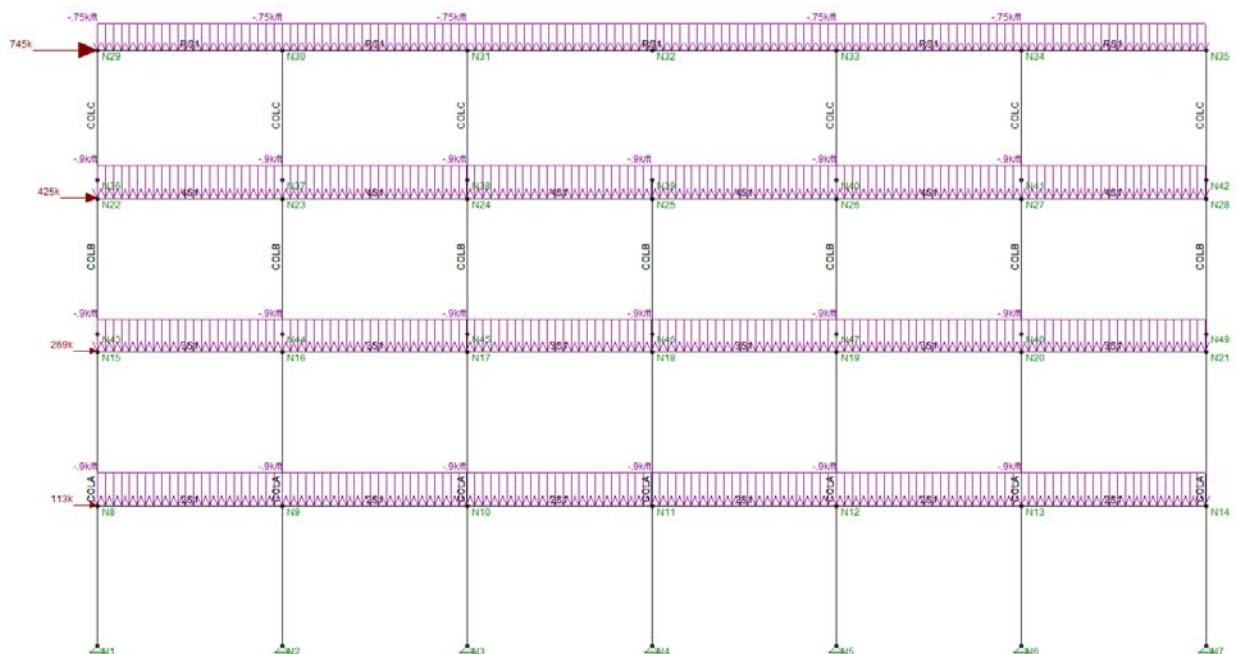
Vertical Diaphragm Distribution (ASCE 41-13 7.4.1.3.4)

Level	w_x	Σw_x	F_x	ΣF_x	F_{px} , ULT
ROOF	1,484,763	1,484,763	1,489,181	1,489,181	1,489,181
4th	1,215,413	2,700,176	849,595	2,338,776	1,052,738
3rd	1,288,088	3,988,264	538,044	2,876,820	929,125
2nd	1,343,798	5,332,062	226,440	3,103,260	782,090
Σ	5,332,062		3,103,260		

N-S Direction Frames



E-W Direction Frames



3325 Chanate Road, Santa Rosa, CA 95404

Drift Check (Sec 4.5.3.1)

Story	Frame N-S (seis)				Frame E-W (seis)			
	Story V	Col V	Drift R*		Col V	Drift R*		
				Life Safety	Immediate Ocp		Life Safety	Immediate Ocp
R	745	165.6	0.026	NOT OK	NOT OK	155.7	0.026	NOT OK
4th	425	221.7	0.015	OK	OK	198.2	0.015	OK
3rd	269	247.6	0.012	OK	OK	246.6	0.012	OK
2nd	113	247.6	0.013	OK	OK	246.6	0.013	OK

* Per RISA Model (EQ only combo).... Gives higher numbers than (Eqn 4-7)

N-S

	Story	Joint(X)	X Drift...	% of Ht
1	2	N8	3.496	2.571
2	2	N15	2.187	1.458
3	2	N22	1.827	1.218
4	2	N29	1.901	1.32

E-W

	Story	Joint(X)	X Drift...	% of Ht
1	2	N8	3.495	2.571
2	2	N15	2.187	1.458
3	2	N22	1.81	1.206
4	2	N29	1.803	1.252

Axial Stress Check (Sec 4.5.3.6)

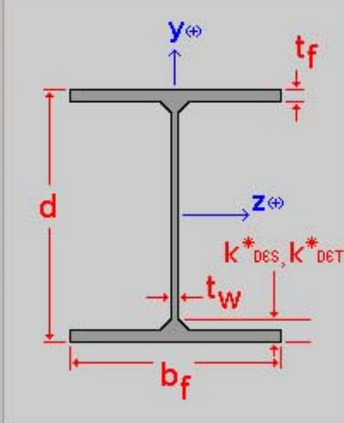
Frame			Stress			
	Col	Allow	IO		LS	
N-S	M5 (1st)	3.6	0.91	OK	0.59	OK
E-W	M5 (1st)	3.6	0.65	OK	0.43	OK
Alternate		Allow	IO		LS	
N-S	M7 (1st)	10.8	9.10	OK	5.91	OK
E-W	M7 (1st)	10.8	9.19	OK	5.97	OK
Per RISA model						
	m=1.3	Immediate Occupancy				
	m=2.0	Life Safety				
Forces per RISA model						

3325 Chanate Road, Santa Rosa, CA 95404

Beam and Column Properties:

Typ Roof Frame Beam:

Edit Shape



Shape Properties

Shape Name: RS1

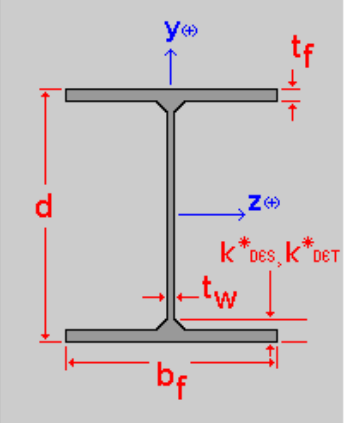
Depth	24	in	Z _{yy}	41.875	in ³
Flange Width	9	in	Z _{zz}	267.5	in ³
Flange Thick	1	in	C _w	16098.682	in ⁶
Web Thick	.5	in	W _{no}	51.75	in ²
Area	29	in ²	S _w	116.438	in ⁴
I _{yy}	121.729	in ⁴	r _T	2.369	in
I _{zz}	2825.667	in ⁴	k* _{des}	0	in
J	6.484	in ⁴	k* _{det}	0	in

k* is for Connection calcs only 'Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

Typ 4th Floor Frame Beam:

Edit Shape



Shape Properties

Shape Name: 4S1

Depth	24	in	Z _{yy}	51.969	in ³
Flange Width	9	in	Z _{zz}	313.719	in ³
Flange Thick	1.25	in	C _w	19680.179	in ⁶
Web Thick	.5	in	W _{no}	51.188	in ²
Area	33.25	in ²	S _w	143.965	in ⁴
I _{yy}	152.099	in ⁴	r _T	2.414	in
I _{zz}	3328.318	in ⁴	k* _{des}	0	in
J	11.576	in ⁴	k* _{det}	0	in

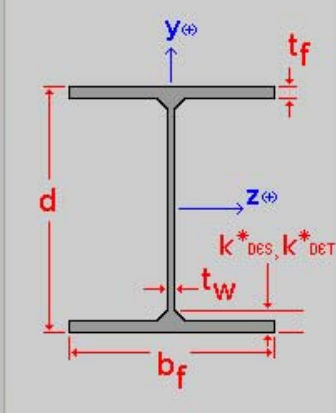
k* is for Connection calcs only 'Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

3325 Chanate Road, Santa Rosa, CA 95404

Typ 3rd Floor Frame Beam:

Edit Shape



Shape Properties

Shape Name: 3S1

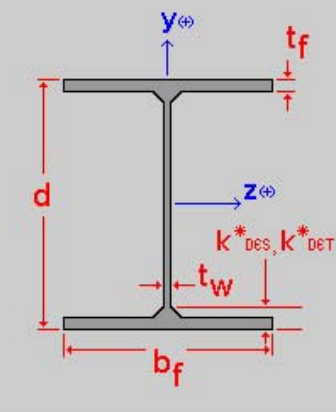
Depth	24	in	Zyy	72.156	in ³
Flange Width	9	in	Zzz	402.969	in ³
Flange Thick	1.75	in	Cw	26342.095	in ⁶
Web Thick	.5	in	Wno	50.063	in ²
Area	41.75	in ²	Sw	197.121	in ⁴
Iyy	212.839	in ⁴	rT	2.468	in
Izz	4265.62	in ⁴	k*des	0	in
J	29.058	in ⁴	k*det	0	in

k* is for Connection calcs only 'Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

Typ 2nd Floor Frame Beam:

Edit Shape



Shape Properties

Shape Name: 2S1

Depth	24	in	Zyy	72.156	in ³
Flange Width	9	in	Zzz	402.969	in ³
Flange Thick	1.75	in	Cw	26342.095	in ⁶
Web Thick	.5	in	Wno	50.063	in ²
Area	41.75	in ²	Sw	197.121	in ⁴
Iyy	212.839	in ⁴	rT	2.468	in
Izz	4265.62	in ⁴	k*des	0	in
J	29.058	in ⁴	k*det	0	in

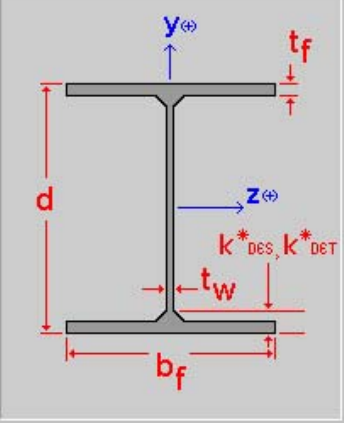
k* is for Connection calcs only 'Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

3325 Chanate Road, Santa Rosa, CA 95404

Typ Frame Column Roof to 4th Floor:

Edit Shape



Shape Properties

Shape Name: COLC

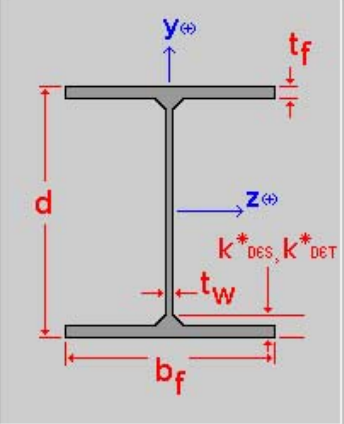
Depth	21 in	Z _{yy}	43.172 in ³
Flange Width	9 in	Z _{zz}	247.688 in ³
Flange Thick	1 in	C _w	12216.797 in ⁶
Web Thick	.75 in	W _{no}	45 in ²
Area	32.25 in ²	S _w	101.25 in ⁴
I _{yy}	122.168 in ⁴	r _T	2.313 in
I _{zz}	2230.188 in ⁴	k* _{des}	0 in
J	8.185 in ⁴	k* _{det}	0 in

k* is for Connection calcs only 'Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

Typ Frame Column 4th Floor to 3rd Floor:

Edit Shape



Shape Properties

Shape Name: COLB

Depth	21 in	Z _{yy}	73.336 in ³
Flange Width	9 in	Z _{zz}	360.609 in ³
Flange Thick	1.75 in	C _w	19754.709 in ⁶
Web Thick	.75 in	W _{no}	43.313 in ²
Area	44.625 in ²	S _w	170.543 in ⁴
I _{yy}	213.24 in ⁴	r _T	2.436 in
I _{zz}	3261.18 in ⁴	k* _{des}	0 in
J	30.612 in ⁴	k* _{det}	0 in

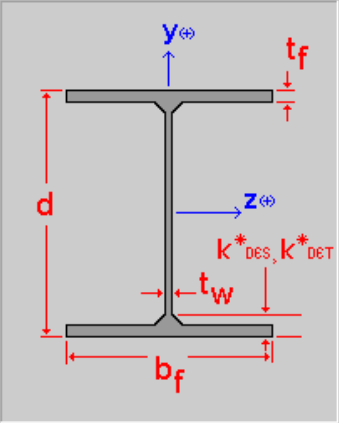
k* is for Connection calcs only 'Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

3325 Chanate Road, Santa Rosa, CA 95404

Typ Frame Column 3rd Floor to Foundation at Ground:

Edit Shape



Shape Properties

Shape Name: COLA

Depth	21	in	Z _{yy}	113.555	in ³
Flange Width	9	in	Z _{zz}	496.734	in ³
Flange Thick	2.75	in	C _w	27866.5	in ⁶
Web Thick	.75	in	W _{no}	41.063	in ²
Area	61.125	in ²	S _w	254.074	in ⁴
I _{yy}	334.67	in ⁴	r _T	2.503	in
I _{zz}	4385.586	in ⁴	k*des	0	in
J	102.874	in ⁴	k*det	0	in

k* is for Connection calcs only 'Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

3325 Chanate Road, Santa Rosa, CA 95404

Flexural Stress Check (Sec 4.5.3.9)

Frame	N-S	Forces per RISA model (EQ only Load combo)					
Member	Top Fb	Bot Fb		max stress	Average Max stress	Fy	
M1	0	0			41.82 ksi	36	NOT COMPLIANT
	32.902	-32.902					
	-10.139	10.139	✓	32.90			
	11.924	-11.924					
	-14.267	14.267					
M2	0	0					
	44.466	-44.466					
	-31.049	31.049	✓	44.47			
	14.218	-14.218					
	-26.071	26.071					
M3	0	0					
	43.829	-43.829					
	-29.521	29.521	✓	43.83			
	14.444	-14.444					
	-25.832	25.832					
M4	0	0					
	43.809	-43.809					
	-29.121	29.121	✓	43.81			
	14.25	-14.25					
	-25.508	25.508					
M5	0	0					
	43.767	-43.767					
	-29.005	29.005	✓	43.77			
	14.276	-14.276					
	-25.821	25.821					
M6	0	0					
	44.241	-44.241					
	-29.838	29.838	✓	44.24			
	13.94	-13.94					
	-25.481	25.481					
M7	0	0					
	32.504	-32.504					
	-9.926	9.926	✓	32.50			
	11.405	-11.405					
	-14.532	14.532					
M8	10.186	-10.186					



Continued for all members

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Flexural Stress Check (Sec 4.5.3.9) cont.

Frame Member	E-W Top Fb	Forces per RISA model (EQ only Load combo) Bot Fb	max stress	Average Max stress	Fy	
M1	0	0		42.76 ksi	36	NOT COMPLIANT
	32.532	-32.532				
	-10.239	10.239	✓ 32.53			
	12.011	-12.011				
	-14.835	14.835				
M2	0	0				
	43.942	-43.942				
	-30.812	30.812	✓ 43.94			
	14.268	-14.268				
	-25.756	25.756				
M3	0	0				
	43.214	-43.214				
	-29.2	29.2	✓ 43.21			
	14.639	-14.639				
	-25.127	25.127				
M4	0	0				
	43.373	-43.373				
	-28.96	28.96	✓ 43.37			
	13.865	-13.865				
	-26.895	26.895				
M5	0	0				
	43.143	-43.143				
	-28.749	28.749	✓ 43.14			
	14.52	-14.52				
	-25.067	25.067				
M6	0	0				
	43.717	-43.717				
	-29.672	29.672	✓ 43.72			
	13.98	-13.98				
	-25.045	25.045				
M7	0	0				
	32.387	-32.387				
	-9.042	9.042	✓ 32.39			
	11.797	-11.797				
	-14.039	14.039				
M8	-19.951	19.951				

**Continued for all members**

3325 Chanate Road, Santa Rosa, CA 95404

Panel Zones (Sec A.3.1.3.5)

Beam one side of column

Columns	dc	wc	tcw*	tcf	Beams	Z	0.8 Mp (Beam)	Shear Demand	Shear Capacity	Comply?
A	20.5	15.5	1.375	2.75	2S1	402.969	11605.51	566.122302	4464.9	Yes
A	20.5	15.5	1.375	2.75	3S1	402.969	11605.51	566.122302	4464.9	Yes
B	21.5	17.5	1.25	1.75	4S1	313.719	9035.107	420.237544	4257	Yes
C	22	19	0.75	1	RS1	267.5	7704	350.181818	2613.6	Yes
*includes doubler plates Using k=1.0										
Beam both sides of column										
Columns	dc	wc	tcw*	tcf	Beams	Z	0.8 Mp (Beam)	Shear Demand	Shear Capacity	Comply?
A	20.5	15.5	1.375	2.75	2S1	402.969	23211.01	1132.2446	4464.9	Yes
A	20.5	15.5	1.375	2.75	3S1	402.969	23211.01	1132.2446	4464.9	Yes
B	21.5	17.5	1.25	1.75	4S1	313.719	18070.21	840.475088	4257	Yes
C	22	19	0.75	1	RS1	267.5	15408	700.363636	2613.6	Yes
*includes doubler plates Using k=1.0										
panel zone m-factor = 8									capacity equation 9-5 ASCE 41-13	
table (9-4) Primary, Life Safety									Per section 9.4.2.4.2 FR conn can be considered	
									Deformation controlled and evaluated using EQ 7-36	
									if not designed to promote yielding of the beam	

Column Splices (Sec 5.5.2.2.3)**Lowest Column Splice Capacity****1) Full Pen Weld...use full section area****Area col B (smaller section in splice)**

A 44.265 in²
Tcap 1593.54 kips

Connection

4 - 2-1/4 A.B.
A 15.9 in²
Tcap 572 kips

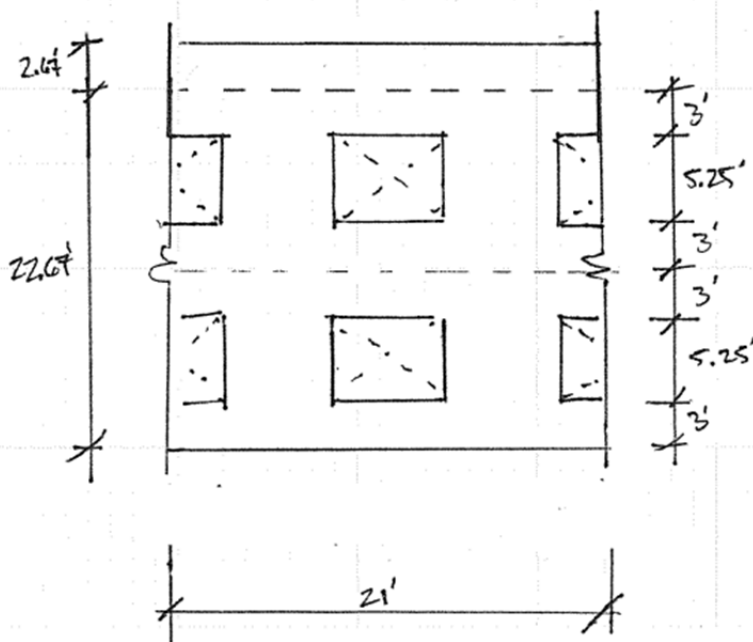
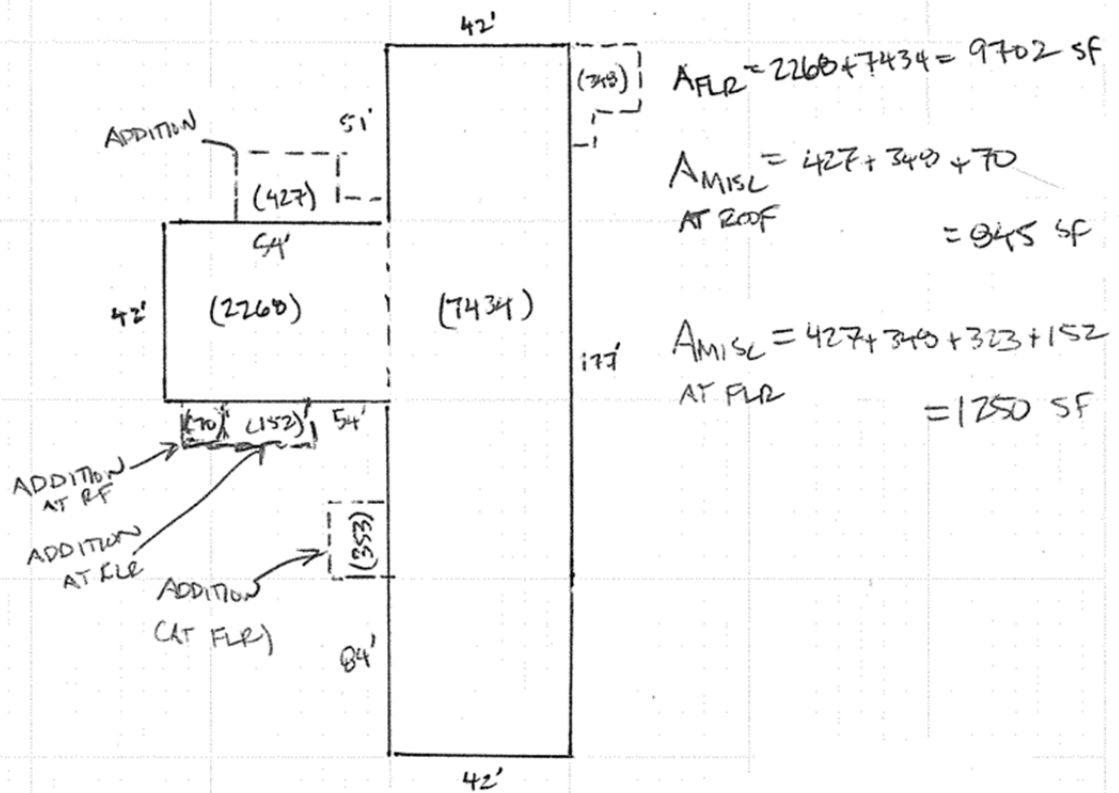
Strong Column/ Weak Beam (Sec A.3.1.3.7)						
N-S End columns						
Columns	Zc	Beams	Zb	col fa	Ratio	Comply
A	469.7	2S1	402.9	12.67	1.32	Yes
A	469.7	3S1	402.9	7.12	1.59	Yes
B	360.6	4S1	313.7	4.82	1.68	Yes
C	247.7	RS1	267.5	2.11	0.73	No
N-S Interior Columns						
Columns	Zc	Beams	Zb	col fa	Ratio	Comply
A	469.7	2S1	402.9	1.469	0.94	No
A	469.7	3S1	402.9	0.955	0.95	No
B	360.6	4S1	313.7	0.88	0.94	No
C	247.7	RS1	267.5	0.59	0.38	No
E-W End Columns						
Columns	Zc	Beams	Zb	col fa	Ratio	Comply
A	469.7	2S1	402.9	11.6	1.37	Yes
A	469.7	3S1	402.9	6.5	1.62	Yes
B	360.6	4S1	313.7	4.4	1.70	Yes
C	247.7	RS1	267.5	1.91	0.73	No
E-W Interior Columns						
Columns	Zc	Beams	Zb	col fa	Ratio	Comply
A	469.7	2S1	402.9	1.192	0.94	No
A	469.7	3S1	402.9	0.706	0.95	No
B	360.6	4S1	313.7	0.651	0.94	No
C	247.7	RS1	267.5	0.428	0.38	No

3325 Chanate Road, Santa Rosa, CA 95404

Compactness (Sec A.3.1.3.8)

Moderately Ductile limits assumed per AISC 341-10 table D1.1							
Beams	b/t	$65/(F_y)^{0.5}$	Compact?	h/t	$253.7/(F_y)^{0.5}$	Compact?	
RS1	9.00	10.83333333	Yes	20.00	42.28333	Yes	
4S1	7.20	10.83333333	Yes	19.00	42.28333	Yes	
3S1	5.14	10.83333333	Yes	17.00	42.28333	Yes	
2S1	5.14	10.83333333	Yes	17.00	42.28333	Yes	
Cols	b/t	$65/(F_y)^{0.5}$	Compact?	b/t	$253.7/(F_y)^{0.5}$	Compact?	
A	3.272727	10.83333333	Yes	13.66667	42.28333	Yes	
B	5.142857	10.83333333	Yes	16.33333	42.28333	Yes	
C	9	10.83333333	Yes	18.33333	42.28333	Yes	

3325 Chanate Road, Santa Rosa, CA 95404

Building 3 (1956 Hospital Wing)1956 ADDITION

BSE-1E

$$S_s = .995$$

$$S_1 = .389$$

$$F_a = 1.002 \rightarrow S_{xs} = .997$$

$$F_v = 1.41 \rightarrow S_{x1} = .548$$

$$T = C_t h_n^{\beta}$$

$$= .02 (22.67')^{.75} = .208$$

$$S_a = \frac{S_{x1}}{T} = \frac{.548}{.21} = 2.637 > S_{xs} \leftarrow \text{USE } S_{xs}$$

$$S_a = .997$$

$$C_m = 1.0 \quad (2\text{-STORY})$$

$$R_{MAX} = 2.5 \quad (\text{SEE FOLLOWING CALCS})$$

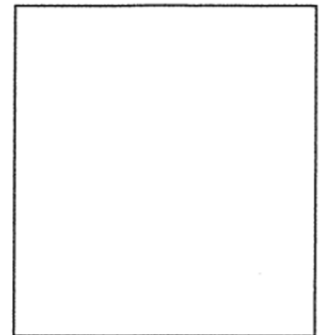
$$C_1 C_2 = 1.4$$

$$C_1 C_2 C_m = 1.4$$

$$V = (1.4)(.997) W$$

$$V = 1.395 W$$

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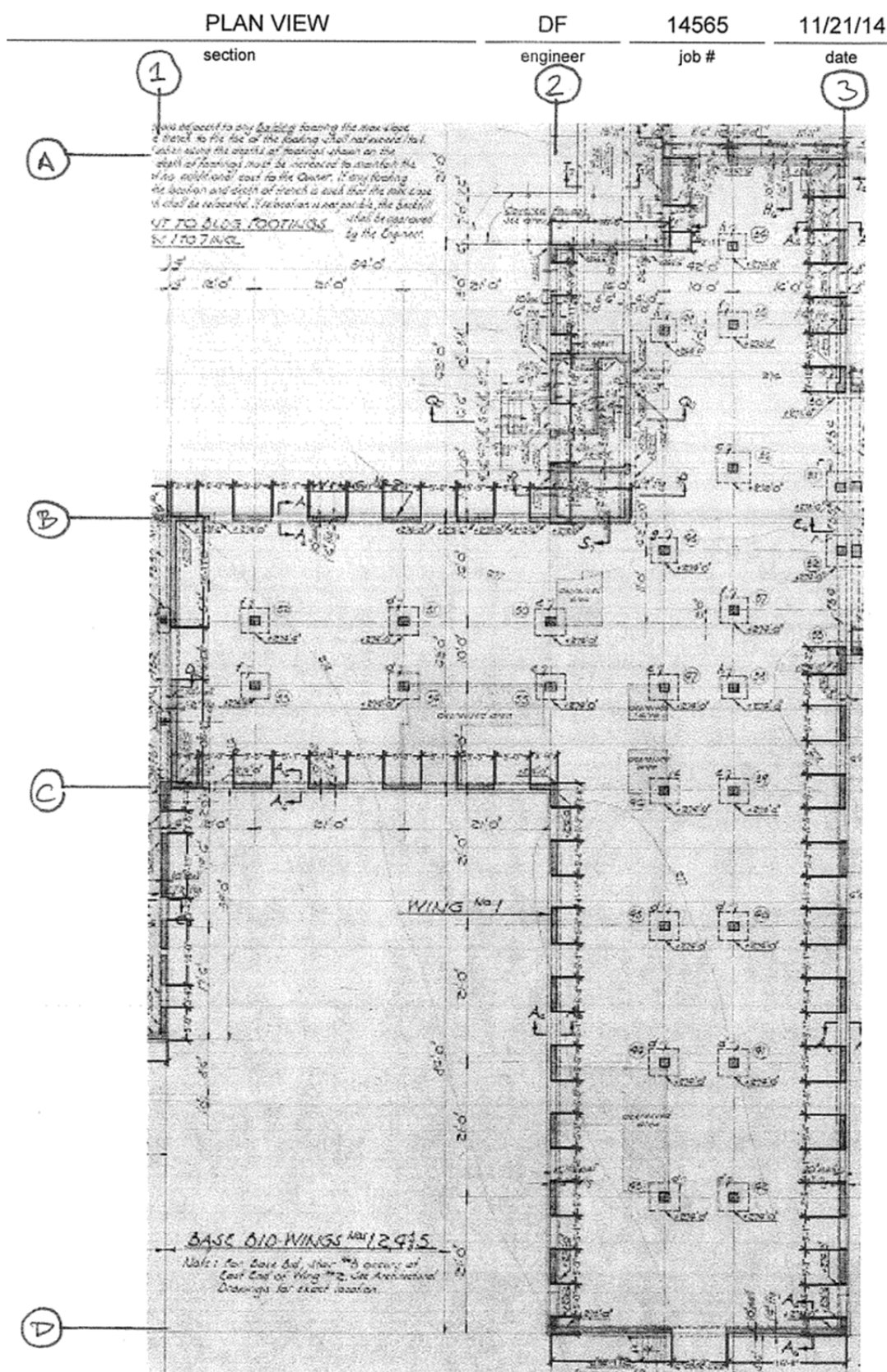
DESIGN CRITERIA**Material (unless noted otherwise)**Concrete: $f_c = 2500$ psi (per existing plans)Reinf. Steel: $f_y = 40000$ psi (per existing plans)

Stamp

DESIGN LOADING

	SLOPED ROOF*	FLAT ROOF*	FLOOR	FLOOR	INTERIOR WALLS	EXTERIOR WALLS
LIVE LOADS (PSF)		20.0	40.0			
DEAD LOADS (PSF)						
Roofing** Light Weight Fill		20.0	0.0			
Fin. Floor		0.0	2.0			
Diaphragm 4.5" RF/6.5" FLR Conc Slab		56.3	81.3			
Joists/Truss		0.0	0.0			
Beams In weight calcs		0.0	0.0			
Ceiling Plaster Ceiling		8.0	8.0			
Insulation		0.0	0.0			
HVAC		2.0	2.0			
Partitions		0.0	10.0			
Sprinklers		1.5	1.5			
Misc.		2.2	2.2			
DEAD LOADS (PSF)	0.0	90.0	107.0	0.0	0.0	125.0
TOTAL LOADS (PSF)	0.0	110.0	147.0	0.0		

3325 Chanate Road, Santa Rosa, CA 95404



LINE 1:

$$\Sigma \text{ Full-HT walls} = 17' + 17' = 34'$$

$$\Sigma \text{ walls w/ openings} = 0'$$

LINE 2:

$$\Sigma \text{ Full-HT walls} = 7.25' + 3' + 267' + 6' + 7.67' + 9.25' + 3.67' + (5.5' \times 7) + 267' = 81'$$

$$\Sigma \text{ walls w/ openings} = 177' - 81' - 40' = 56'$$

LINE 3:

$$\Sigma \text{ Full-HT walls} = (5.5' \times 12) + 2.67' + (37.5' \times 2) = 76'$$

$$\Sigma \text{ walls w/ openings} = 177' - 76' - 40' = 61'$$

LINE A:

$$\Sigma \text{ Full-HT walls} = 17' + 16.5' = 33.5'$$

$$\Sigma \text{ walls w/ openings} = 8.5'$$

LINE B:

$$\Sigma \text{ Full-HT walls} = 367' + (5.5' \times 4) + 14' = 40'$$

$$\Sigma \text{ walls w/ openings} = 5 \times 5' = 25'$$

LINE C:

$$\Sigma \text{ Full-HT walls} = 3.75' + (5.5' \times 4) + 3.75' = 29.5'$$

$$\Sigma \text{ walls w/ openings} = 5' \times 5' = 25'$$

LINE D:

$$\Sigma \text{ Full-HT walls} = 17' + 17' = 34'$$

$$\Sigma \text{ walls w/ openings} = 0'$$

3325 Chanate Road, Santa Rosa, CA 95404

BLDG 3 - 1956 ADDITION WEIGHTS

Floors	Weight (psf)	Areas	Misc Area	DL (KIP)
Roof	90	9702	845	949
2nd	107	9702	1250	1172

Roof Walls	Weight (psf)	Height	Length	DL (KIP)
LINE 1	125	8.33	34	35.4
LINE 2	125	8.33	81	84.3
LINE 3	125	8.33	76	79.1
LINE A	125	8.33	33.5	34.9
LINE B	125	8.33	40	41.7
LINE C	125	8.33	29.5	30.7
LINE D	125	8.33	34	35.4
TOTAL =				341.5

Roof Walls w/ Opngs	Weight (psf)	Height	Length	DL (KIP)
LINE 1	125	8.33	8	8.3
LINE 2	125	8.33	56	58.3
LINE 3	125	8.33	61	63.5
LINE A	125	8.33	8.5	8.9
LINE B	125	8.33	25	26.0
LINE C	125	8.33	25	26.0
LINE D	125	8.33	0	0.0
TOTAL =				191.1

2nd Floor Walls	Weight (psf)	Height	Length	DL (KIP)
LINE 1	125	11.33	34	48.2
LINE 2	125	11.33	81	114.7
LINE 3	125	11.33	76	107.6
LINE A	125	11.33	33.5	47.4
LINE B	125	11.33	40	56.7
LINE C	125	11.33	29.5	41.8
LINE D	125	11.33	34	48.2
TOTAL =				464.5

2nd Floor Walls w/ Opngs	Weight (psf)	Height	Length	DL (KIP)
LINE 1	125	11.33	8	11.3
LINE 2	125	11.33	56	79.3
LINE 3	125	11.33	61	86.4
LINE A	125	11.33	8.5	12.0
LINE B	125	11.33	25	35.4
LINE C	125	11.33	25	35.4
LINE D	125	11.33	0	0.0
TOTAL =				259.9

Columns	Weight (psf)	Height	Quantity	DL (KIP)
Roof	267	5.66	22	33
2nd	267	11.33	22	67

Beams	Weight (psf)	Length	Quantity	DL (KIP)
Roof	175	231	2	81
2nd	233	231	2	108

Grand Total Dead Loads Per Floor		DL (KIP)	Floor Area	KSF/FLR
	Roof	1596	9702	0.164
	2nd	2155	9703	0.222
	Total	3,751k		
	V =	1.395W		

Vertical Distribution of Seismic Forces	Height	Weight	w _i *h _i	w _i *h _i /sum(w _i h _i)	FX	Force per floor (kips)
Roof	22.66	1596	36163.68803	0.60	3124	3124
2nd	11.33	2155	24414.06012	0.40	2109	2109
		3751	60577.74816	1		5232

3325 Chanate Road, Santa Rosa, CA 95404

CENTER OF MASS

Building Overall Dimensions	X=	96.0 ft.
	Y=	177.0 ft.

ITEM	WEIGHT	cmx	cmy	W*cmx	W*cmy
SLAB A	547	27	105	14769	57435
SLAB B	1611	75	89	120825	142574
WALL LINE 1	103.0	0.0	84.0	0	8652
WALL LINE 2	337.0	54.0	31.5	18198	10616
WALL LINE 3	337.0	96.0	52.5	32352	17693
WALL LINE A	103.0	75.0	177.0	7725	18231
WALL LINE B	160.0	27.0	126.0	4320	20160
WALL LINE C	134.0	27.0	84.0	3618	11256
WALL LINE D	84.0	75.0	0.0	6300	0
				0	0
	3416			208107	286616

CMX=	60.9 ft.
------	----------

CMY=	83.9 ft.
------	----------

Accidental torsion, 5% ex= 4.8 ey= 8.9

Amplification Factor, Ax Ax = 1.0 Ax = 1.0

Amplified eccentricity Amp ex= 4.8 Amp ey= 8.9

CMX + Amp ex=	65.7 ft.
CMX - Amp ex=	56.1 ft.

CMY + Amp ey=	92.8 ft.
CMY - Amp ey=	75.1 ft.

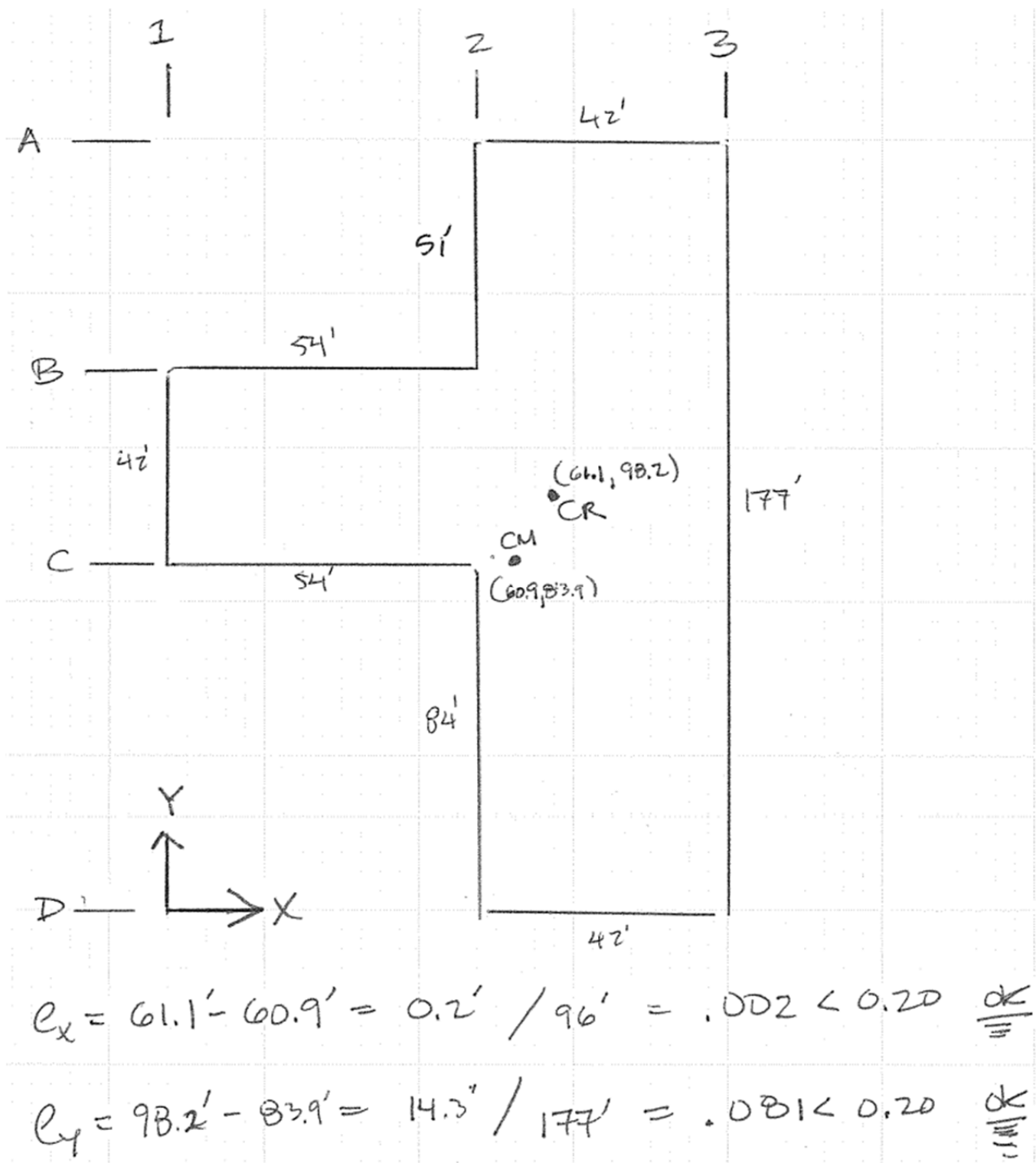
CENTER OF RIGIDITY**CRY (WALLS IN THE X DIRECTION)**

WALL	Description	Rx	Y	Rx*Y
A		33.50	177.00	5929.50
B		40.00	126.00	5040.00
C		29.50	84.00	2478.00
D		34.00	0.00	0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
SUM		137.00		13447.50
			CRY=	98.2 ft.

CRX (WALLS IN THE Y DIRECTION)

WALL	Description	Ry	X	Ry*X
1		34.00	0.00	0.00
2		81.00	54.00	4374.00
3		76.00	96.00	7296.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
SUM		191.00		11670.00
			CRX=	61.1 ft.

3325 Chanate Road, Santa Rosa, CA 95404



3325 Chanate Road, Santa Rosa, CA 95404

HORIZONTAL DISTRIBUTION

FORCE DIRECTION =	X	(ROOF)
V =	3124 kips	

CMY + Ey 5 ft.
M = V*e = 16879 kip-ft

CMY - Ey 23 ft.
M = V*e = 72174 kip-ft

WALL	Rx		Y	D	R*D	R*D^2	Fv	Fm @ + Ey	Fm @ - Ey	Ftotal
WALLS IN THE X DIRECTION										Max
A	33.5		177.0	-78.8	-2641.2	208243.7	763.9	-56.0	-239.4	707.9
B	40.0		126.0	-27.8	-1113.7	31009.5	912.1	-23.6	-100.9	888.5
C	29.5		84.0	14.2	417.6	5912.4	672.7	8.9	37.9	710.5
D	34.0		0.0	98.2	3337.3	327582.6	775.3	70.7	302.5	1077.8
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Rx= 137.0

572748.1

WALL		Ry	X	D	R*D	R*D^2	Fv	Fm @ + Ey	Fm @ - Ey	Ftotal
WALLS IN THE Y DIRECTION										max
1		34.0	0.0	61.1	2077.4	126927.0	0.0	44.0	188.3	188.3
2		81.0	54.0	7.1	575.1	4082.6	0.0	12.2	52.1	52.1
3		76.0	96.0	-34.9	-2652.4	92571.5	0.0	-56.2	-240.4	-240.4
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Ry= 191.0

223581.1

SUM OF RD^2= 796329.2

→ WALL LINE D CARRIES "SECONDARY FRAME"
AND HAS MAX EQL LOAD (K/FT)

3325 Chanate Road, Santa Rosa, CA 95404

HORIZONTAL DISTRIBUTION

FORCE DIRECTION =	Y	(ROOF)
V =	3124 kips	

CMX + Ex -5 ft.

M = V*e = -14438 kip-ft

CMX - Ex 5 ft.

M = V*e = 15552 kip-ft

WALL	Rx		Y	D	R*D	R*D^2	Fv	Fm @ + Ex	Fm @ - Ex	Ftotal
WALLS IN THE X DIRECTION										Max
A	33.5		177.0	-78.8	-2641.2	208243.7	0.0	47.9	-51.6	-51.6
B	40.0		126.0	-27.8	-1113.7	31009.5	0.0	20.2	-21.8	-21.8
C	29.5		84.0	14.2	417.6	5912.4	0.0	-7.6	8.2	8.2
D	34.0		0.0	98.2	3337.3	327582.6	0.0	-60.5	65.2	65.2
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
SUM OF Rx=	137.0					572748.1				

WALL		Ry	X	D	R*D	R*D^2	Fv	Fm @ + Ex	Fm @ - Ex	Ftotal
WALLS IN THE Y DIRECTION										Max
1		34.0	0.0	61.1	2077.4	126927.0	556.1	-37.7	40.6	596.7
2		81.0	54.0	7.1	575.1	4082.6	1324.8	-10.4	11.2	1336.1
3		76.0	96.0	-34.9	-2652.4	92571.5	1243.1	48.1	-51.8	1291.1
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
SUM OF Ry=		191.0				223581.1				

SUM OF RD^2= 796329.2

3325 Chanate Road, Santa Rosa, CA 95404

HORIZONTAL DISTRIBUTION

FORCE DIRECTION =	X	(FLOOR)
V =	2109 kips	

$$\begin{aligned} \text{CMY} + \text{Ey} &= 5 \text{ ft.} \\ \text{M} = \text{V} \cdot \text{e} &= 11395 \text{ kip-ft} \end{aligned}$$

$$\begin{aligned} \text{CMY} - \text{Ey} &= 23 \text{ ft.} \\ \text{M} = \text{V} \cdot \text{e} &= 48724 \text{ kip-ft} \end{aligned}$$

WALL	Rx		Y	D	R*D	R*D^2	Fv	Fm @ + Ey	Fm @ - Ey	Ftotal
WALLS IN THE X DIRECTION										Max
A	33.5		177.0	-78.8	-2641.2	208243.7	515.7	-37.8	-161.6	477.9
B	40.0		126.0	-27.8	-1113.7	31009.5	615.8	-15.9	-68.1	599.8
C	29.5		84.0	14.2	417.6	5912.4	454.1	6.0	25.6	479.7
D	34.0		0.0	98.2	3337.3	327582.6	523.4	47.8	204.2	727.6
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Rx= 137.0

572748.1

WALL		Ry	X	D	R*D	R*D^2	Fv	Fm @ + Ey	Fm @ - Ey	Ftotal
WALLS IN THE Y DIRECTION										max
1		34.0	0.0	61.1	2077.4	126927.0	0.0	29.7	127.1	127.1
2		81.0	54.0	7.1	575.1	4082.6	0.0	8.2	35.2	35.2
3		76.0	96.0	-34.9	-2652.4	92571.5	0.0	-38.0	-162.3	-162.3
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Ry= 191.0

223581.1

SUM OF RD^2= 796329.2

3325 Chanate Road, Santa Rosa, CA 95404

HORIZONTAL DISTRIBUTION

FORCE DIRECTION =	Y	(FLOOR)
V =	2109 kips	

CMX + Ex -5 ft.

M = V*e = -9747 kip-ft

CMX - Ex 5 ft.

M = V*e = 10499 kip-ft

WALL	Rx		Y	D	R*D	R*D^2	Fv	Fm @ + Ex	Fm @ - Ex	Ftotal
WALLS IN THE X DIRECTION										Max
A	33.5		177.0	-78.8	-2641.2	208243.7	0.0	32.3	-34.8	-34.8
B	40.0		126.0	-27.8	-1113.7	31009.5	0.0	13.6	-14.7	-14.7
C	29.5		84.0	14.2	417.6	5912.4	0.0	-5.1	5.5	5.5
D	34.0		0.0	98.2	3337.3	327582.6	0.0	-40.9	44.0	44.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	98.2	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Rx= 137.0

572748.1

WALL		Ry	X	D	R*D	R*D^2	Fv	Fm @ + Ex	Fm @ - Ex	Ftotal
WALLS IN THE Y DIRECTION										Max
1		34.0	0.0	61.1	2077.4	126927.0	375.4	-25.4	27.4	402.8
2		81.0	54.0	7.1	575.1	4082.6	894.4	-7.0	7.6	902.0
3		76.0	96.0	-34.9	-2652.4	92571.5	839.2	32.5	-35.0	871.6
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	61.1	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Ry= 191.0

223581.1

SUM OF RD^2= 796329.2

3325 Chanate Road, Santa Rosa, CA 95404

SW CHECK

section

DF

engineer

14565

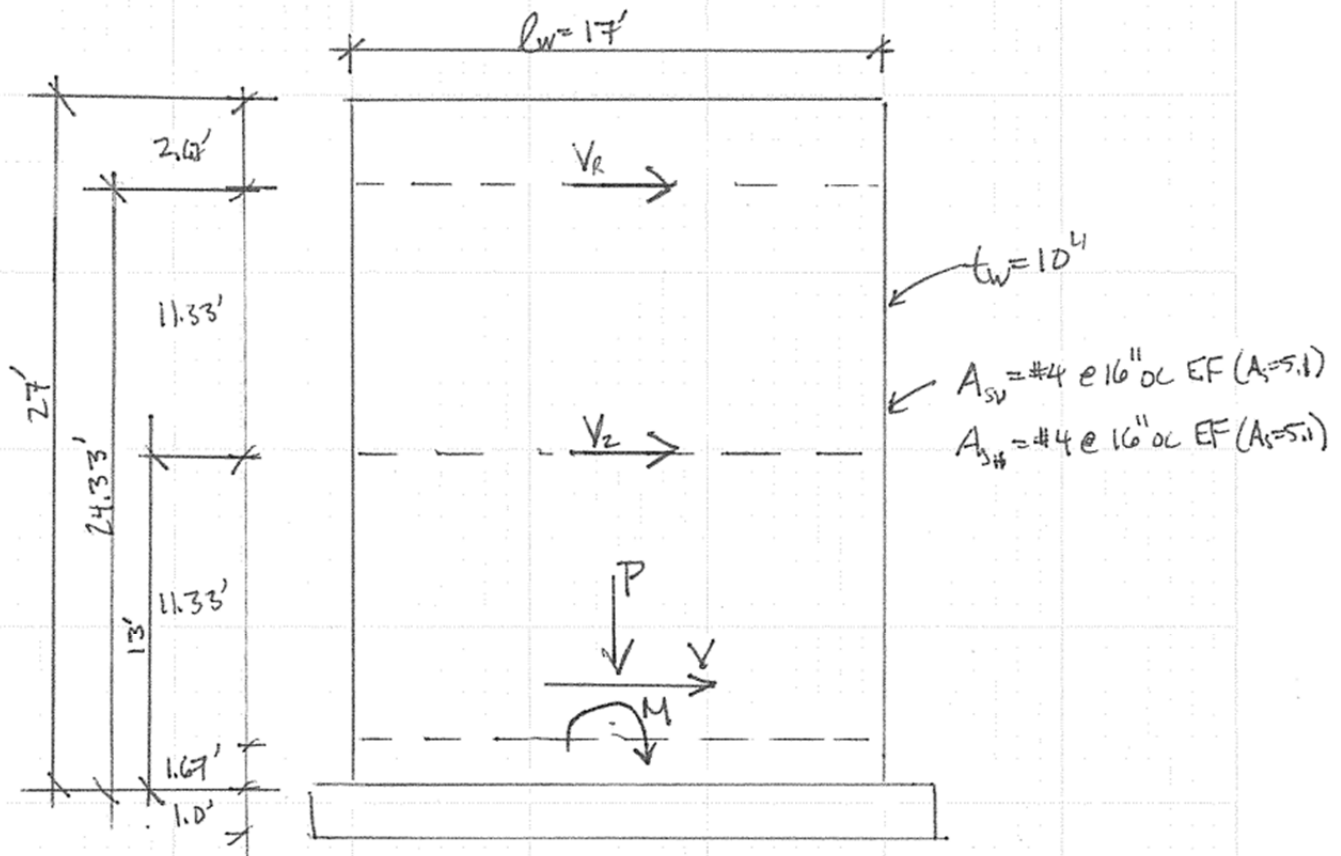
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11/14

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SW ELEVATION



$$P = (90 \text{ PSF})(21' / 2)(17') + (107 \text{ PSF})(21' / 2)(17') + (10'' / 12 \times 150 \text{ PCF})(27')(17') = 92.5^k$$

$$V_R = 538.9^k$$

$$V_2 = 363.8^k$$

$$V = 538.9^k + 363.8^k = 902.7^k$$

$$M = (538.9^k)(24.33') + (363.8^k)(13') - (92.5^k)(17' / 2) = 17,055^k\text{-ft}$$

3325 Chanate Road, Santa Rosa, CA 95404

DESIGN CRITERIA FOR SHEAR WALLS

$$K = 0.9$$

$$f'_c = 2500 \text{ psi}$$

$$f'_{ce} = 2500 \times 1.5 = 3750 \text{ psi}$$

← TABLE 10-1

$$f_y = 40,000 \text{ psi}$$

$$f_{ye} = 40,000 \times 1.25 = 50,000 \text{ psi}$$

← TABLE 10-1

TABLE 10-21

$$\frac{(A_s - A'_s) f_y + P}{t_w l_w f'_c} = \frac{(.5 \times 5.1)(40) + (.925)}{(10)(17 \times 12)(2.5)} = .04 \leq 0.1$$

^{F (ASSUMED)}

$$\frac{V}{t_w l_w \sqrt{f'_c}} = \frac{9027 \times 1000}{(10)(17 \times 12)(\sqrt{2500})} = 8.8 > 6$$

$$.15 A_g f'_c = .15 \times 10' \times 17' \times 12' / 4 \times 2.5$$

$$= 765 \text{ K} >> P_{\max}$$

[CONTROLLED BY
FLEXURE]

$$m = 2.5$$

CHECK SHEAR IN WALL

$$Q_{CE} = 2 \sqrt{f'_c} h d + \frac{(A_{sw})(f_y)(d)}{(s)}$$

$$d = .8 l_w \text{ (ACI 11.9.4)}$$

$$d = .8 (17' \times 12) = 163.2''$$

$$Q_{CE} = 2(\sqrt{3750})(10'')(163.2'') + \frac{(.40)(50 \text{ ksi})(163.2'')}{(16'')}$$

$$Q_{CE} = 200^k + 204^k = 404^k$$

$$M K Q_{CE} = (2.5)(1.9)(404^k) = 909^k > 902.7^k \quad \underline{\underline{OK}}$$

3325 Chanate Road, Santa Rosa, CA 95404

CHECK FLEXURE IN WALL

- ASSUME TENSION CONTROLLED (NOT COMPRESSION CONTROLLED)

$$w = \frac{A_s f_y}{f'_c l_w t} = \frac{(5.1)(50)}{(3.75)(17 \times 12)(10)} = .033$$

$$\alpha = \frac{P_u}{f'_c l_w t} = \frac{(92.5)}{(3.75)(17 \times 12)(10)} = .012$$

$$\frac{c}{l_w} = \frac{w + \alpha}{2w + \alpha \beta_1} = \frac{.033 + .012}{2(.033) + (.012 \times .85)} = .06$$

$$M_n = .5 A_s f_y l_w \left(1 + \frac{P}{A_s f_y}\right) \left(1 - \frac{c}{l_w}\right)$$

$$M_n = (5)(5.1)(50)(17 \times 12) \left(1 + \frac{92.5}{(5.1 \times 50)}\right) (1 - .06)$$

$$M_n = 33,318 \text{ k-ft}$$

$$m K Q_{CE} = (2.50)(.9)(33,318) = 74,966 \text{ k-ft} > 17,055 \text{ k-ft}$$

OK

3325 Chanate Road, Santa Rosa, CA 95404

TIER 1
MISC CALLS:SHKAR WALLS:

$$V_j^{AVG} = \frac{1}{M_s} \left(\frac{V_j}{A_w} \right) \rightarrow \text{CHECK WORST CASE DIR AT BASE}$$

$$A_w = (10\frac{1}{2}) (84' + 108') (.5) = 80 \text{ SF}$$

↑
DOORS

$$v = \left(\frac{1}{4.0} \right) \left(\frac{3815^k \times 1000}{80 \times 144 / \text{ft}^2} \right) = 85 \text{ psi} < 100 \text{ psi} \quad \text{OK}$$

WALL REINF:

#4 @ 16" O.C. EW EF (TYP)

$$A_s = .2 \times 12" / 16" \text{ O.C.} \times 2 \text{ FACES} = .30 \text{ in}^2 / \text{ft}$$

$$\rho = \frac{.30 \text{ in}^2}{10" \times 12} = .0025 > .0020 \text{ (H)} \\ > .0012 \text{ (V)}$$

OK
✓COUPLING BEAMS:

$$d_{TYP} = 66"$$

$$d/2 = 66" / 2 = 33" > 16" \text{ O.C. SPCG}$$



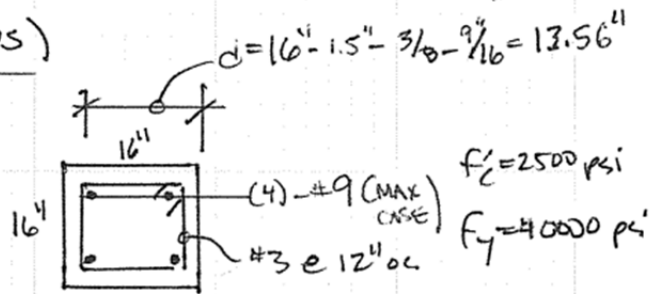
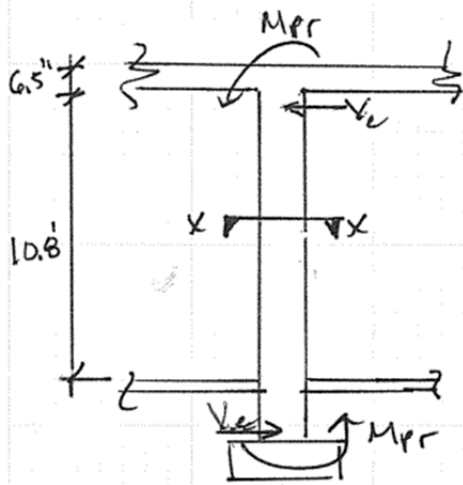
← NO 135° HOOKS ON STIRRUPS

N.G.
✓ADJACENT BLDGS:

$$22.67' \times .04 = 10.9" > 8"$$

N.G.
✓

3325 Chanate Road, Santa Rosa, CA 95404

DEFLECTION COMPATABILITY (COLUMNS)

$$V_e = \frac{M_{pr} - (-M_{pr})}{L} \leq V_n$$

$$M_{pr} = 1.25 A_s f_y (d - a/2)$$

$$a = \frac{(2 \times 1.0)(40,000)}{(0.85)(2500)(16)} = 2.35$$

$$M_{pr} = 1.25 (2 \times 1.0)(40)(13.63" - 2.35"/2) = 1246 \text{ K-in}$$

$$\frac{M_{pr} - (-M_{pr})}{L} = \frac{1246 \text{ K-in} \times 2}{10.8' \times 12} = 19 \text{ K}$$

$$V_n = V_s + V_c \rightarrow 0 \text{ (NEGLECT CONCR DURING EQ EVENT)}$$

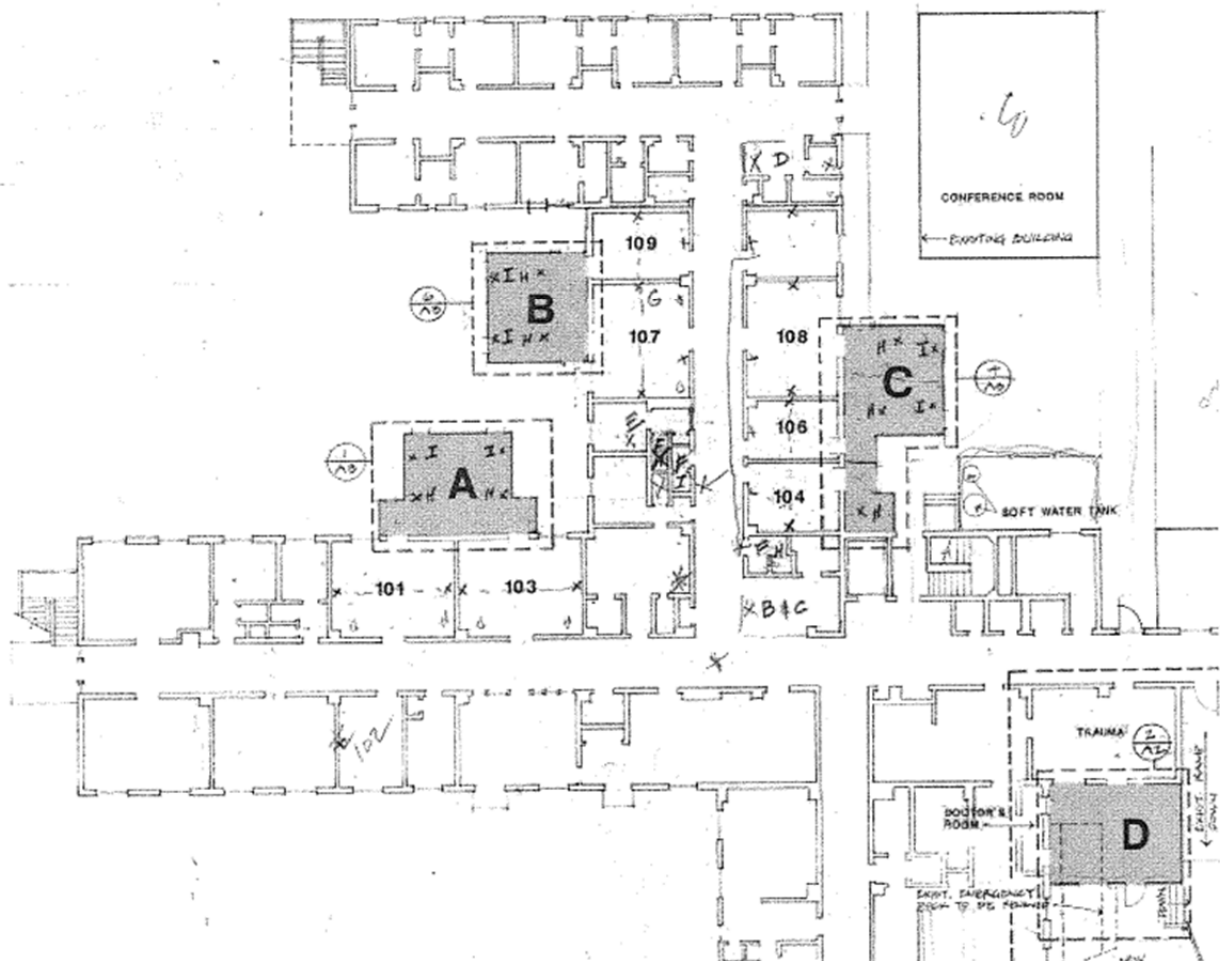
$$V_s = \frac{A_v f_y d}{s} = \frac{(.11 \text{ in}^2 \times 2)(40 \text{ ksi})(13.56")}{4"} = 29.9 \text{ K} > V_e$$

4" \nwarrow SPCC IN TOP 12" OF COLUMN

OK

Building 3 (Steel Appendages) (1956 Hospital Wing)

STEEL APPENDAGES... SITE PLAN DF 14565 12/14
 section engineer job # date page



BSE-1E

$$S_s = .995$$

$$S_1 = .389$$

$$F_a = 1.002 \rightarrow S_{x5} = .997$$

$$F_v = 1.41 \rightarrow S_{x1} = .548$$

$$T = C_t h_n^{\beta}$$

$$= .035 (22.67')^{.80} = .425$$

$$S_a = \frac{S_{x1}}{T} = \frac{.548}{.425} = 1.289 > S_{x5} \leftarrow \text{USE } S_{x5}$$

$$S_a = .997$$

$$C_m = 1.0 \text{ (2-STORY)}$$

$$M_{min} < 2$$

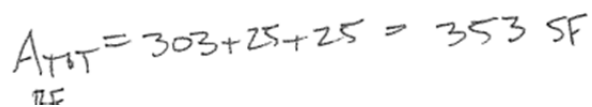
$$C_1 C_2 = 1.1$$

$$C_1 C_2 C_m = 1.1$$

$$V = (1.1)(.997)(W)$$

$$V = 1.097 W$$

page



3325 Chanate Road, Santa Rosa, CA 95404

ZFA STRUCTURAL ENGINEERS

Building 3 – Area B Addition

project name

ROOF PLAN VIEW

DF

14565

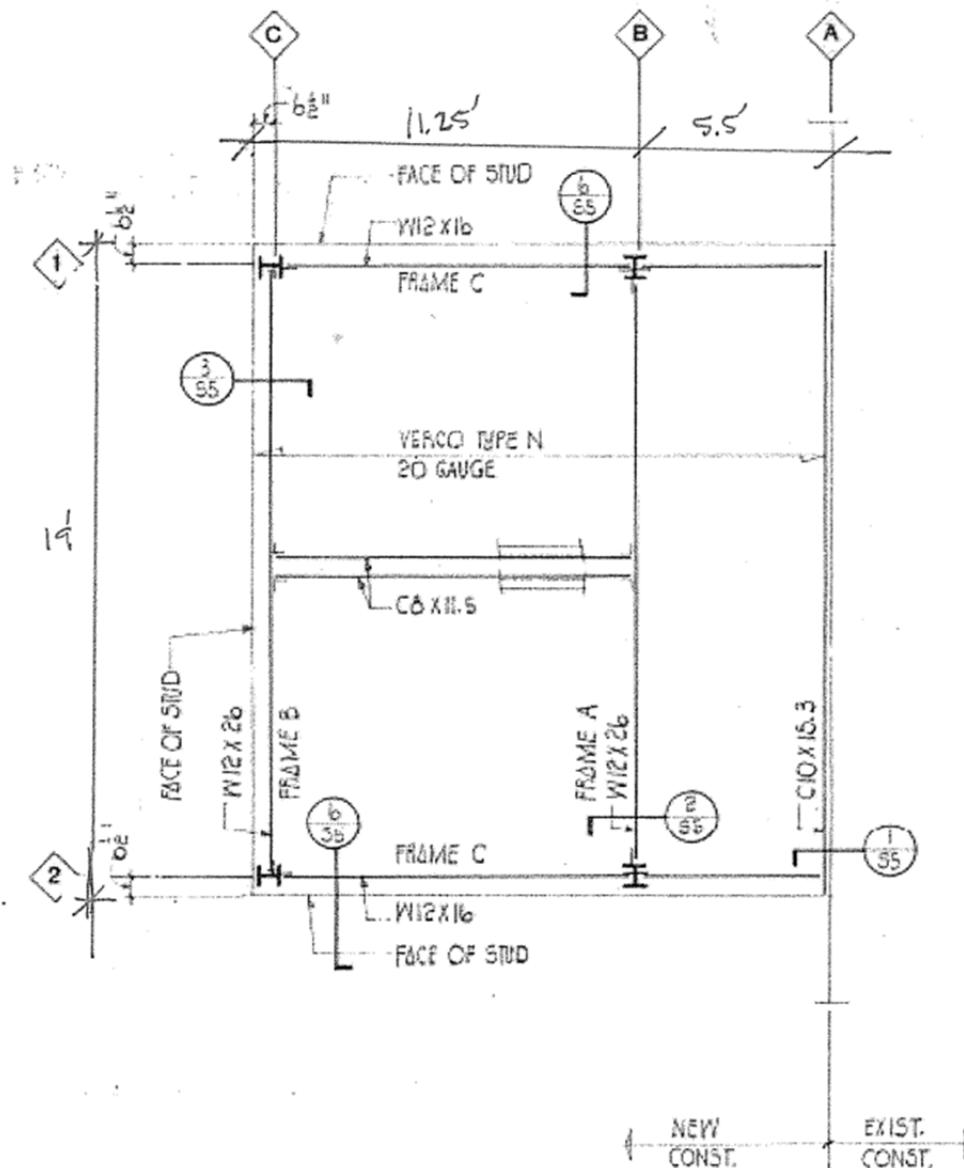
12/14

section

engineer

job #

date



$$A_{TOT} = 310 \text{ SF}$$

RF

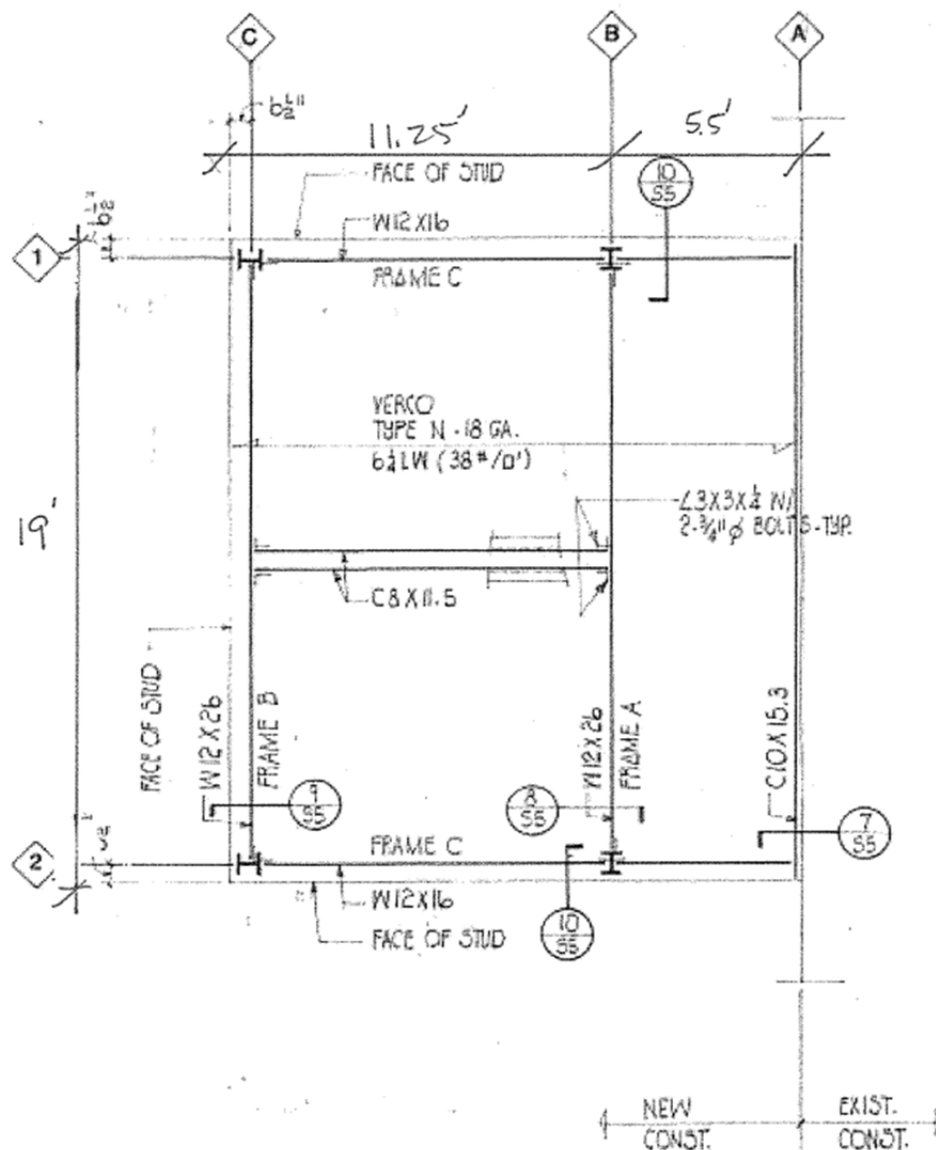
DF

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engineer

job #

date



$$A_{TOT} = 310 \text{ SF}$$

ZFA STRUCTURAL ENGINEERS

Building 3 – Area C Addition

project name

ROOF PLAN VIEW

DF

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12/14

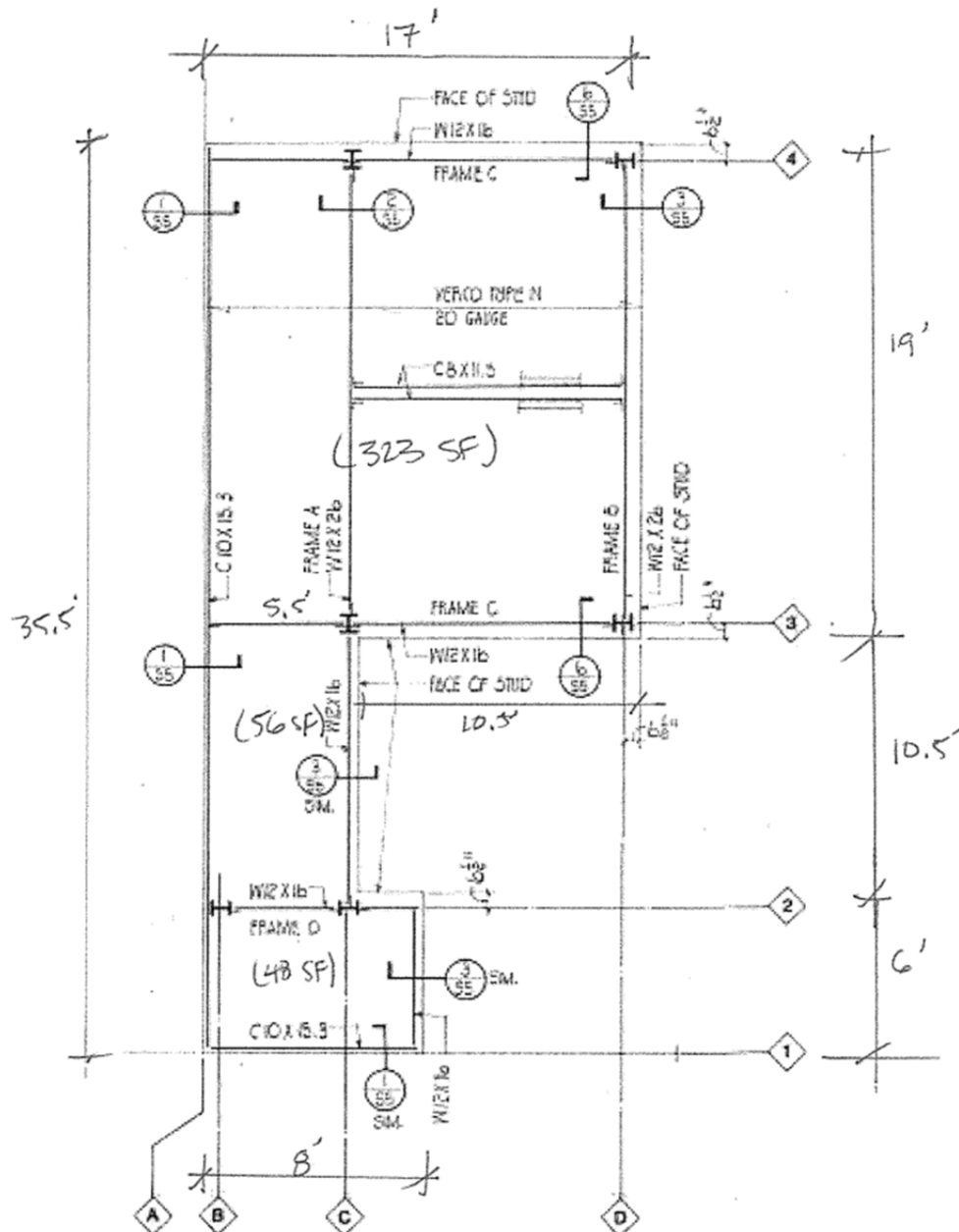
section

engineer

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date

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$$A_{TOT} = 323 + 56 + 48 = 427 \text{ SF}$$

RF

(MAX STEEL ADDITION)
- GWS MF CHECKS

ZFA STRUCTURAL ENGINEERS

Building 3 – Area C Addition

project name

FLOOR PLAN VIEW

DF

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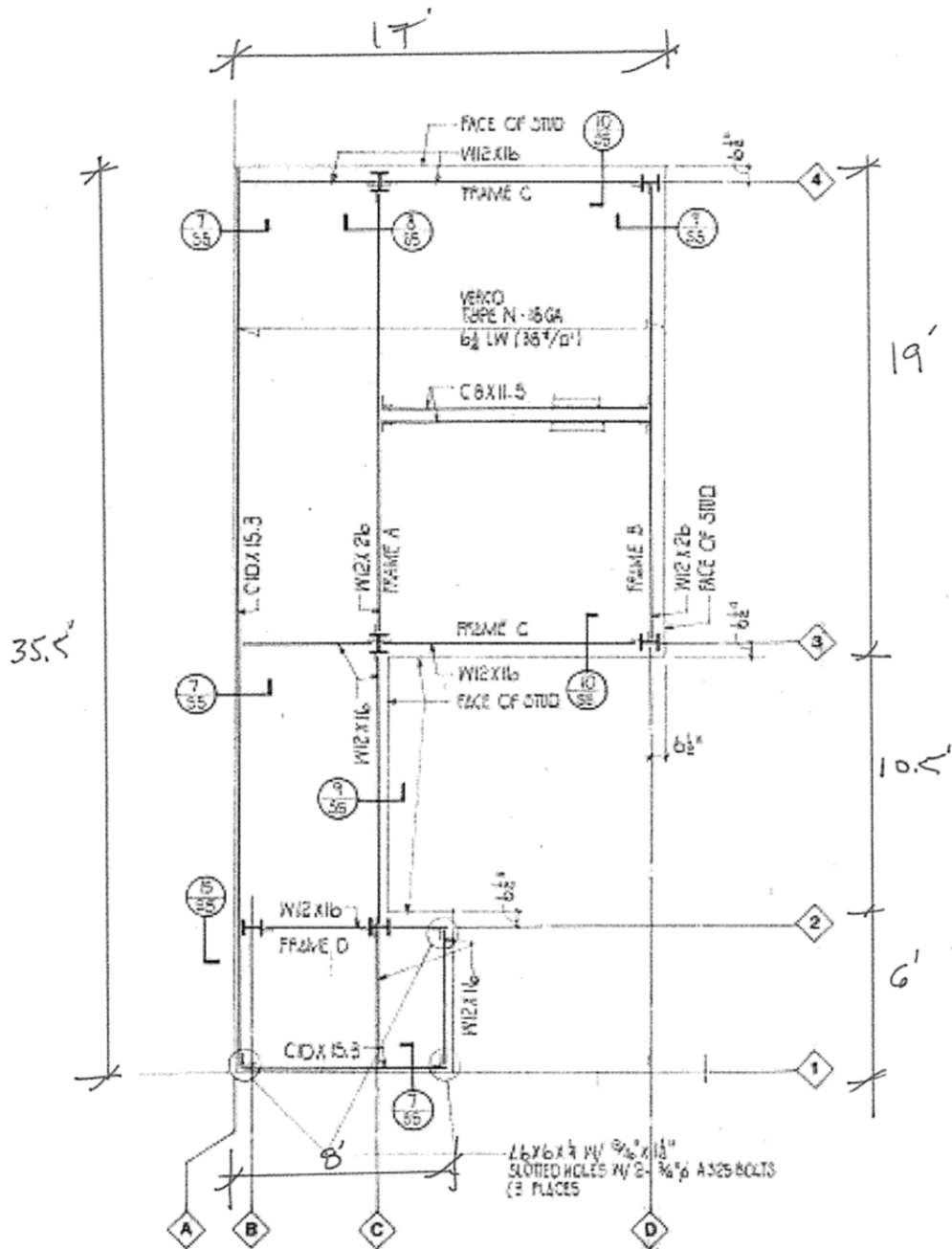
section

engineer

job #

date

page



$$A_{TOT} = 323 + 56 + 48 = 427 \text{ SF}$$

FLR

(MAX STEEL ADDITION)
- GDN'S MF CHECKS

ZFA STRUCTURAL ENGINEERS

Building 3 – Area D Addition

project name

ROOF PLAN VIEW

DF

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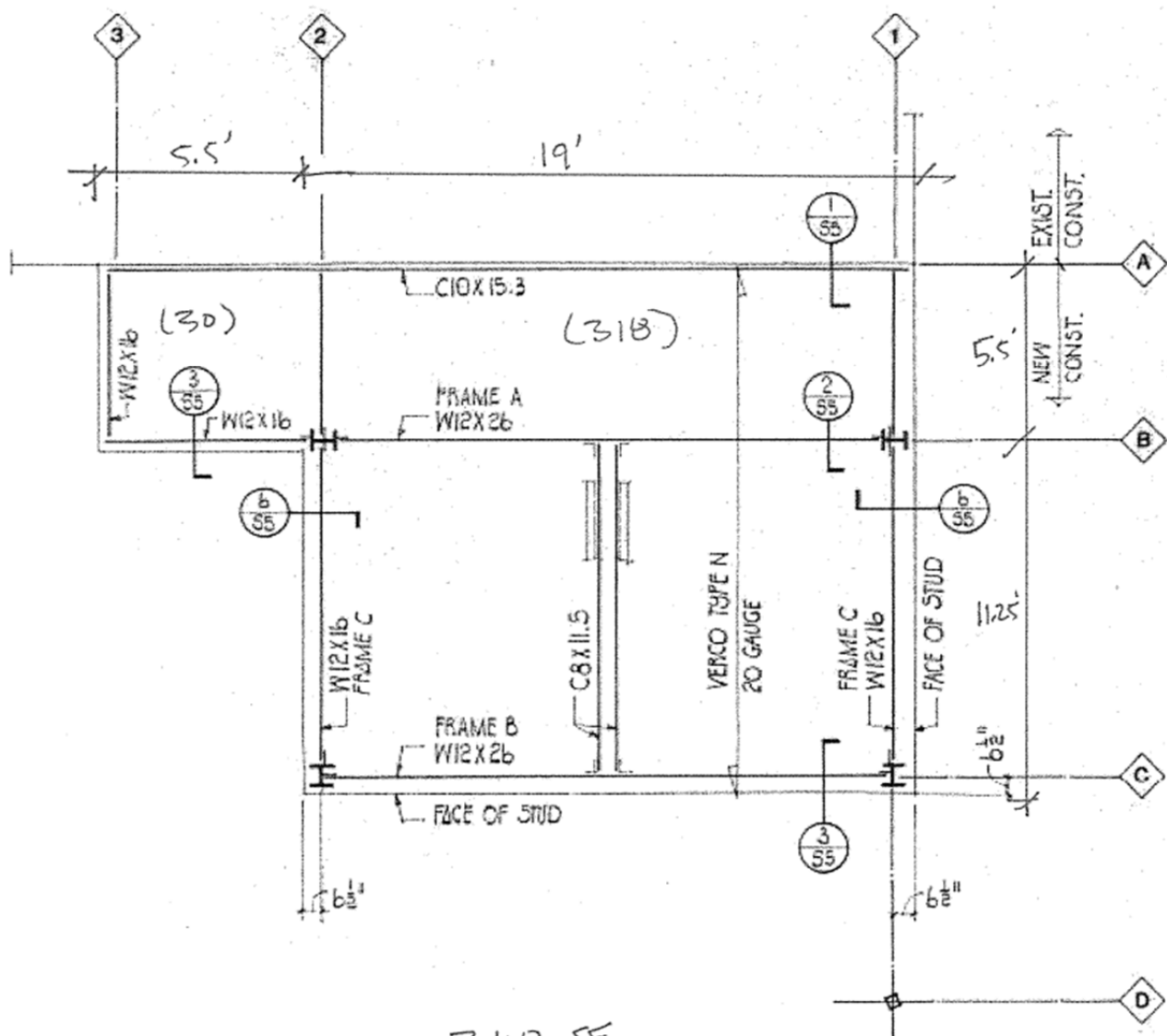
section

engineer

job #

date

page



$$A_{TOT} = 31B + 30 = 349 \text{ SF}$$

RF

ZFA STRUCTURAL ENGINEERS

Building 3 – Area D Addition

project name

FLOOR PLAN VIEW

DF

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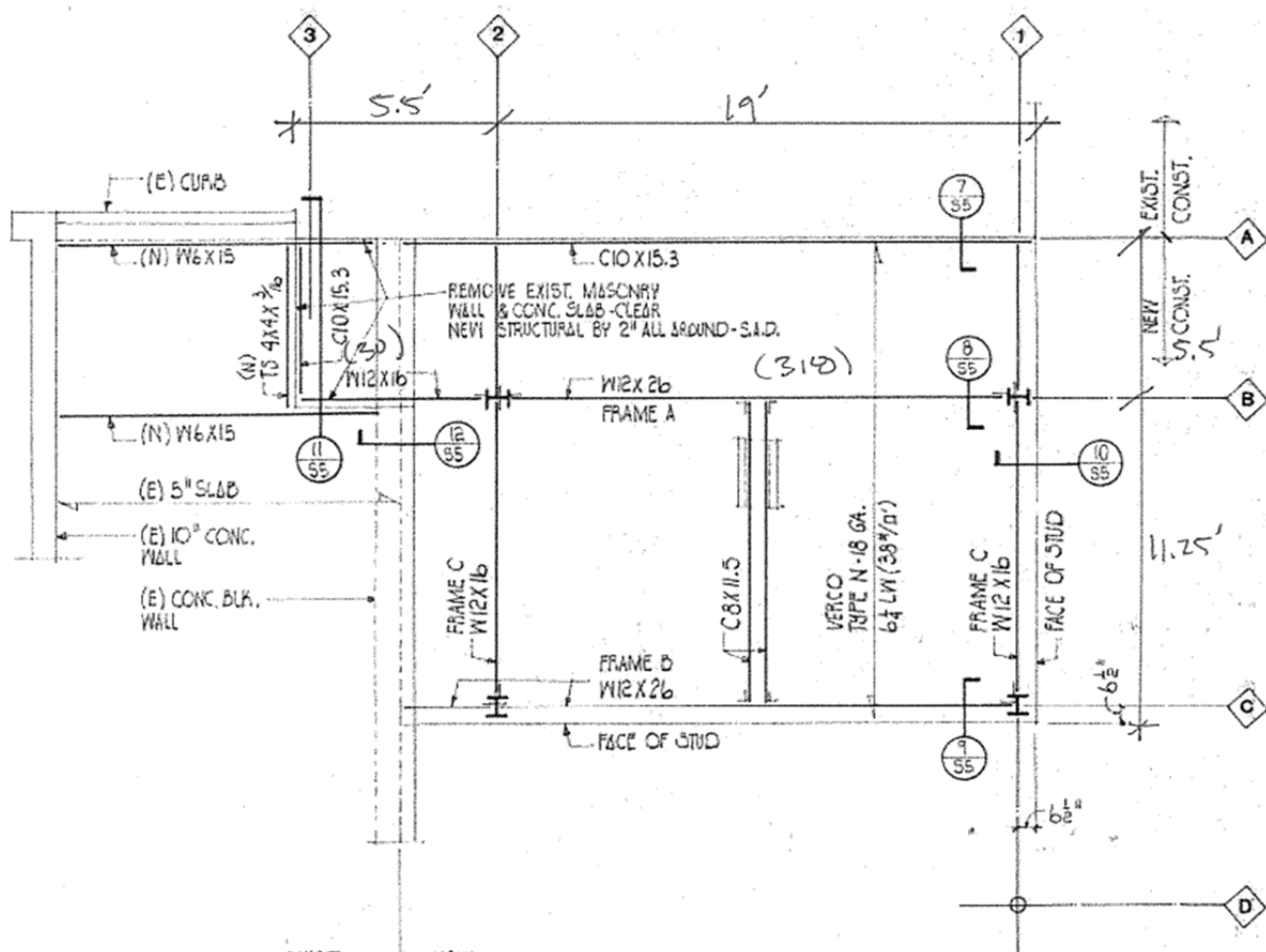
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$$A_{TOT} = 318 + 30 = 348 \text{ SF}$$

FLR

3325 Chanate Road, Santa Rosa, CA 95404

BLDG 3 - AREA C STEEL ADDITION WEIGHTS

Floors	Weight (psf)	Areas	Misc Area	DL (KIP)
Roof	22	427	0	9
2nd	60	427	0	26

Roof Walls	Weight (psf)	Height	Length	DL (KIP)
LINE 1	15	8.33	8	1.0
LINE 2	15	8.33	3	0.4
LINE 3	15	8.33	10.5	1.3
LINE 4	15	8.33	17	2.1
LINE A	15	8.33	0	0.0
LINE B	15	8.33	0	0.0
LINE C	15	8.33	16.5	2.1
LINE D	15	8.33	19	2.4
TOTAL =				9.2

2nd Floor Walls	Weight (psf)	Height	Length	DL (KIP)
LINE 1	15	11.33	8	1.4
LINE 2	15	11.33	3	0.5
LINE 3	15	11.33	10.5	1.8
LINE 4	15	11.33	17	2.9
LINE A	15	11.33	0	0.0
LINE B	15	11.33	0	0.0
LINE C	15	11.33	16.5	2.8
LINE D	15	11.33	19	3.2
TOTAL =				12.6

Columns	Weight (plf)	Height	Quantity	DL (KIP)
Roof	35	5.66	6	1
2nd	35	11.33	6	2

Grand Total Dead Loads Per Floor				DL (KIP)	Floor Area	KSF/FLR
Roof				20	427	0.046
2nd				41	427	0.095
Total				61k		
V =				1.097W		

3325 Chanate Road, Santa Rosa, CA 95404

Seismic Story Force Distribution based on ASCE 41-13

Ta Period (7.4.1.2.2)= 0.425

k= 1

(7.4.1.3.2)

V(ULT)= 1.097

Base V (ULT)= 66,917

Story Force Vertical Distribution (ASCE 41-13 7.4.1.3.2)

Level	w_x	h_x (ft.)	h_x^k	$w_x h_x^k$	F_x , ULT	Load to frame	$Cv_x\%$
ROOF	20,000	23.7	23.7	473400	33,775	16,887	50.5
2nd	41,000	11.3	11.3	464530	33,142	16,571	49.5
Σ	61,000			937930	66,917		

Vertical Diaphragm Distribution (ASCE 41-13 7.4.1.3.4)

Level	w_x	Σw_x	F_x	ΣF_x	F_{px} , ULT
ROOF	20,000	20,000	33,775	33,775	33,775
2nd	41,000	61,000	33,142	66,917	44,977
Σ	61,000		66,917		

DF

14565

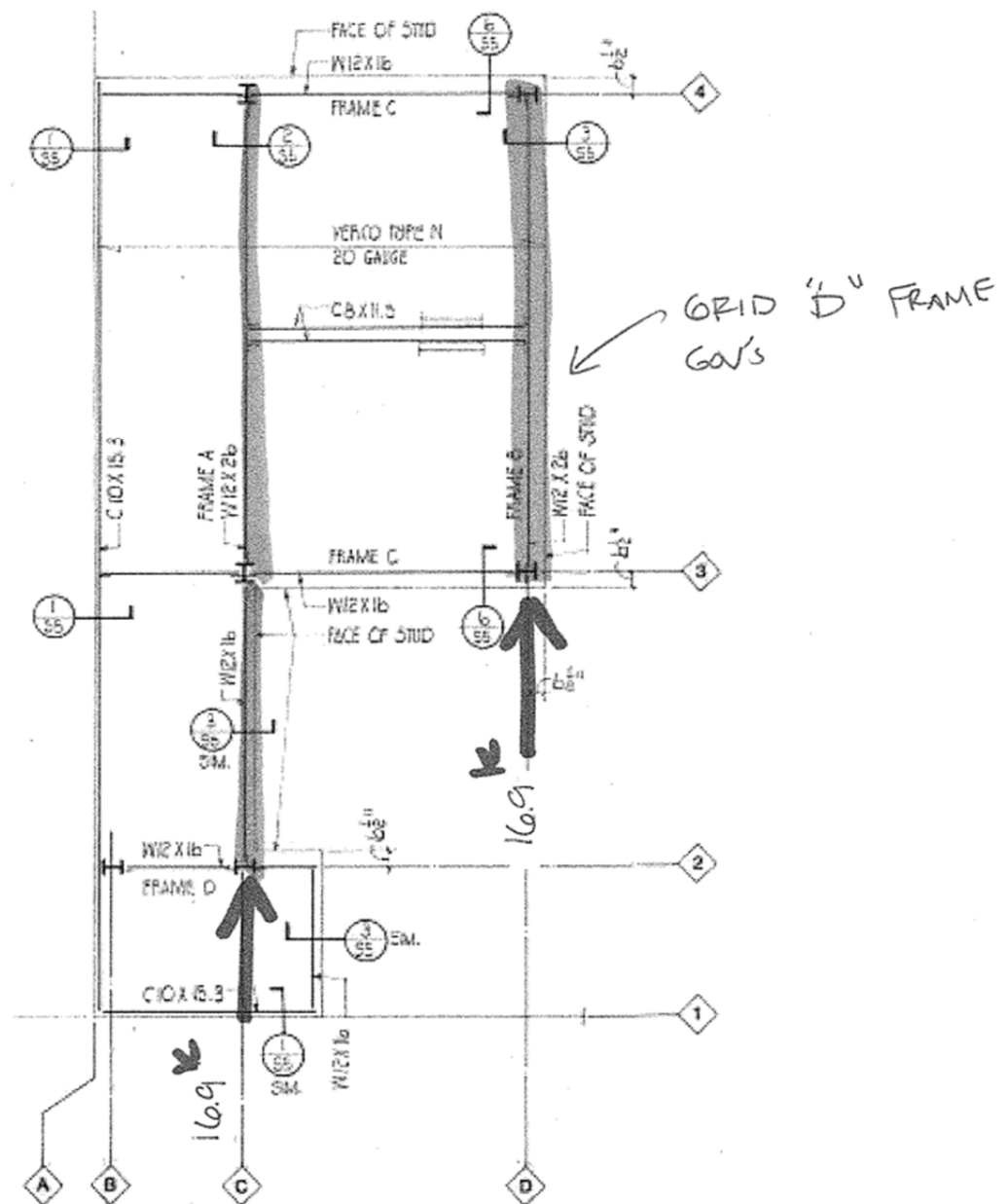
12/14

engineer

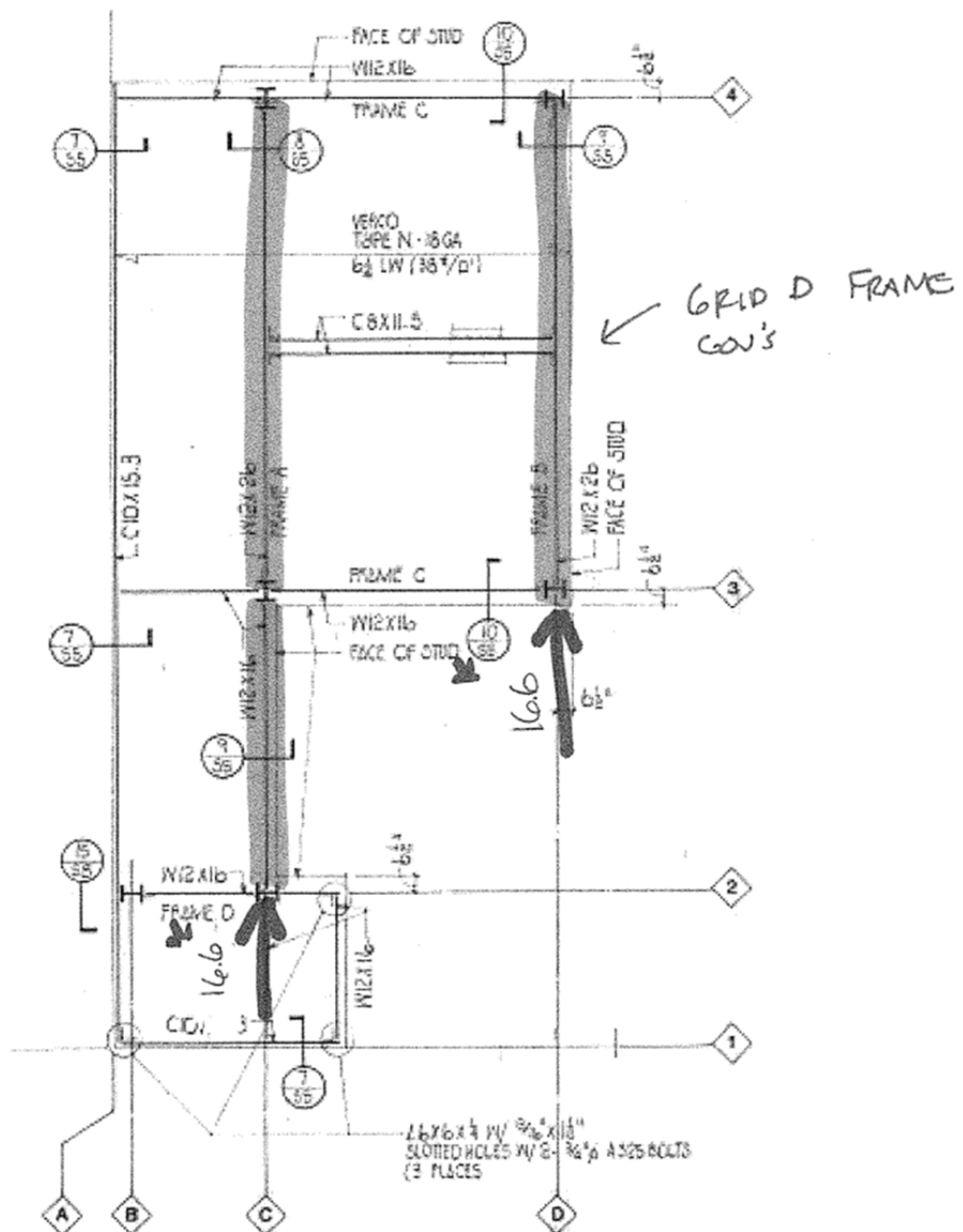
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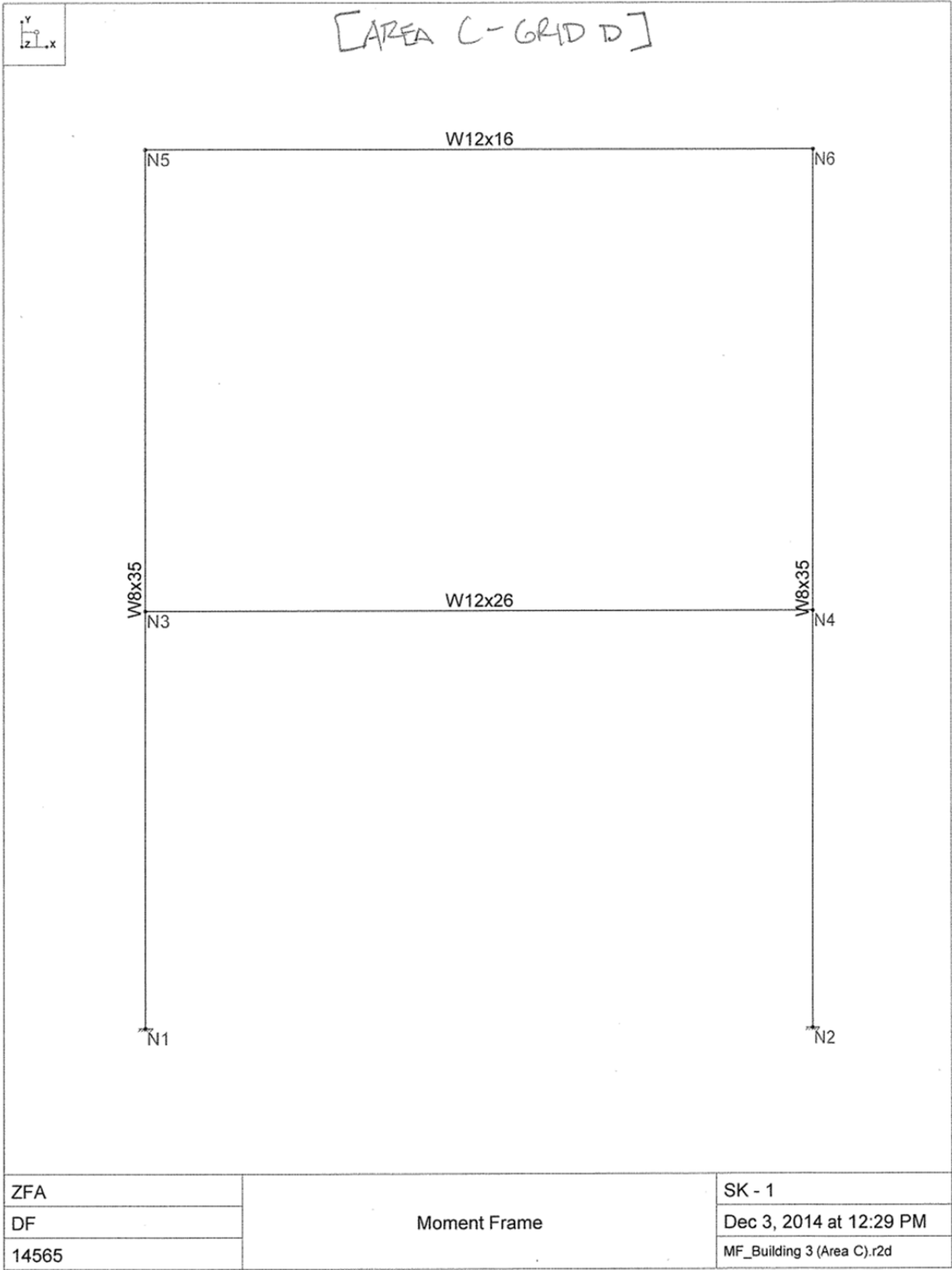
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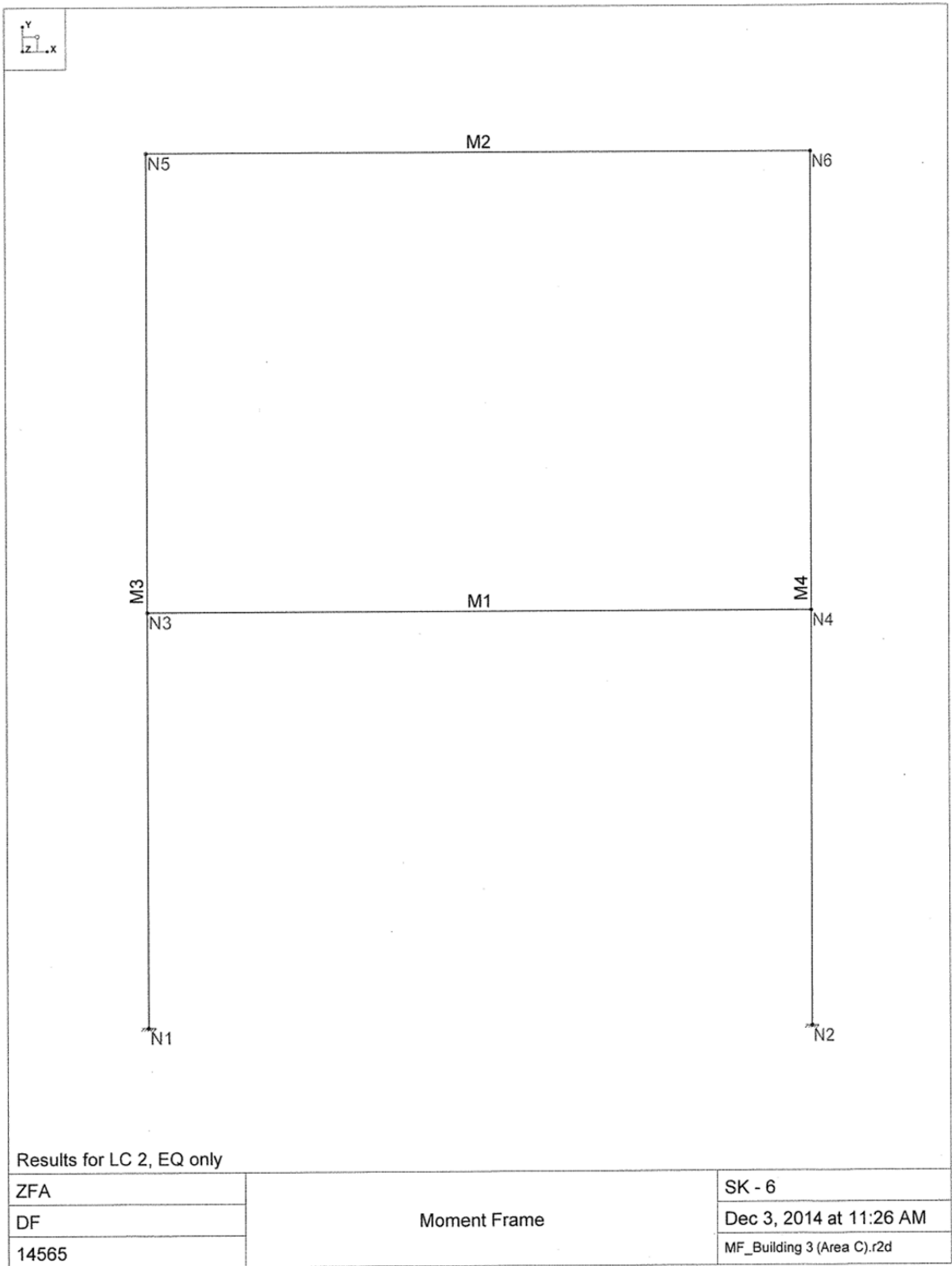
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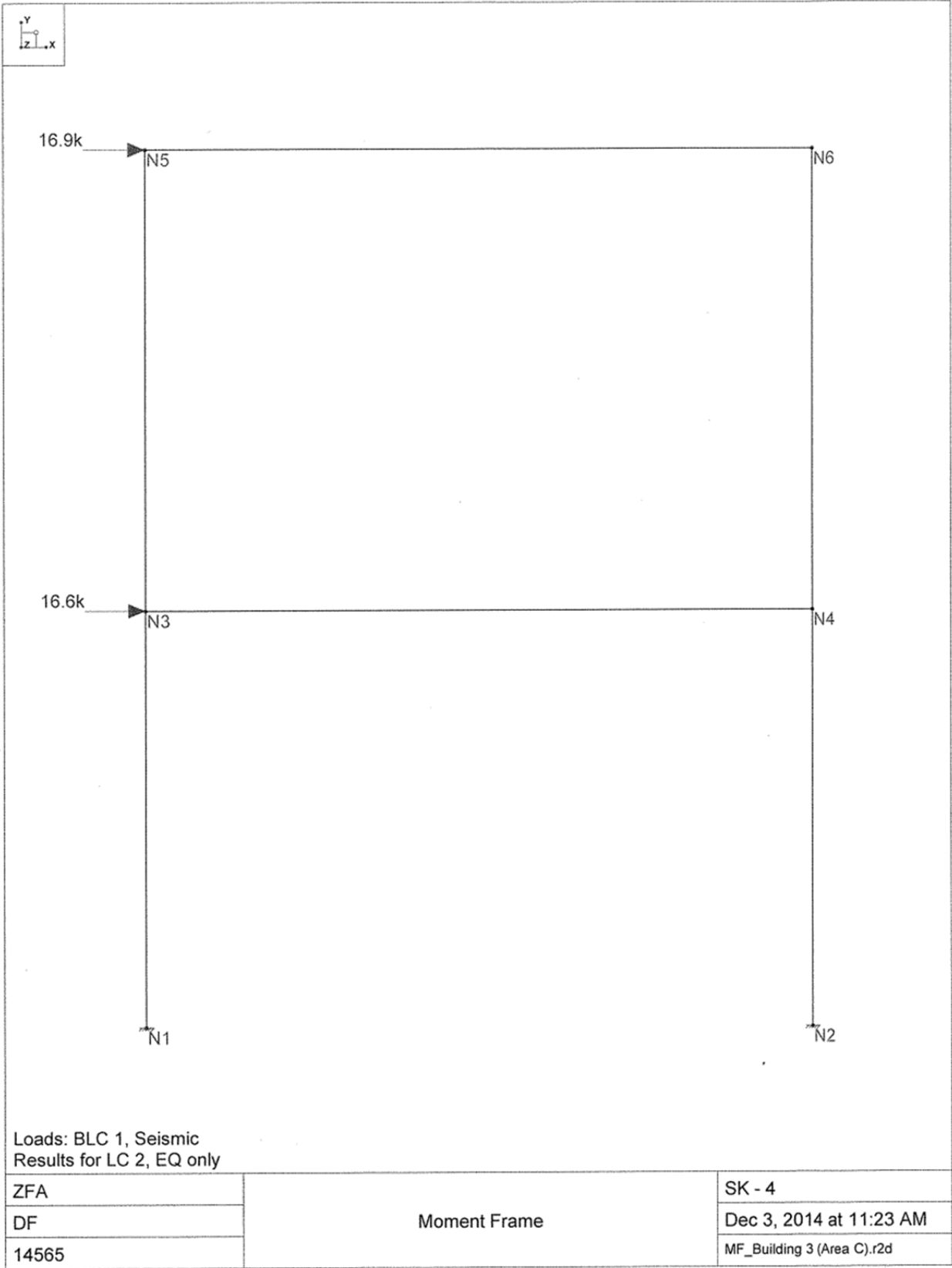
3325 Chanate Road, Santa Rosa, CA 95404



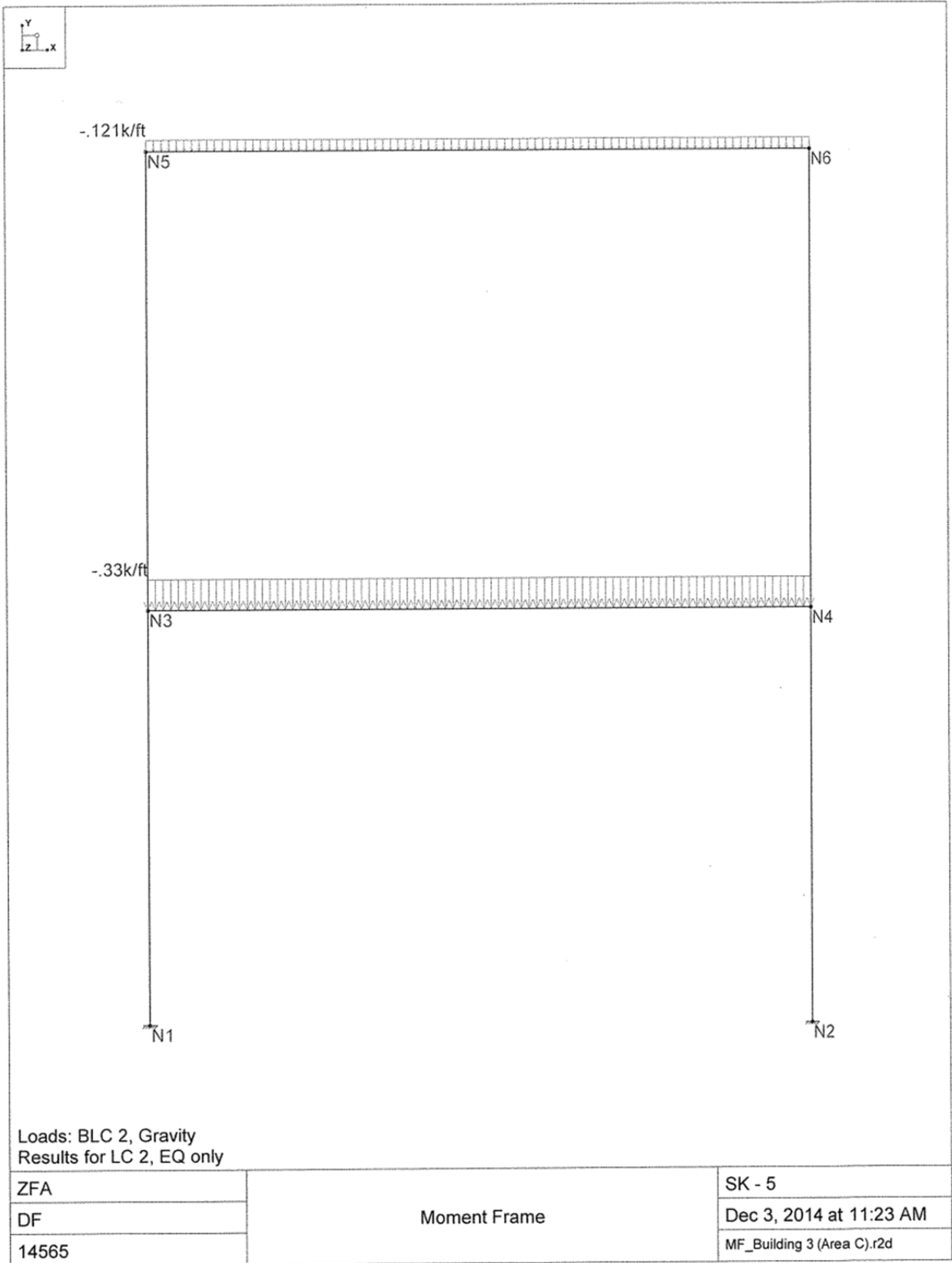
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Company : ZFA
 Designer : DF
 Job Number : 14565
 Model Name : Moment Frame

Dec 3, 2014

Checked By: _____

Member Section Stresses

LC		Member Label	Sec	Axial[ksi]	Shear[ksi]	Top Bending[ksi]	Bot Bending[ksi]
1	2	M1	1	.972	-5.439	49.534	-49.534
2			2	.972	-5.439	24.779	-24.779
3			3	.972	-5.439	.023	-.023
4			4	.972	-5.439	-24.732	24.732
5			5	.972	-5.439	-49.488	49.488
6	2	M2	1	1.736	-2.282	38.094	-38.094
7			2	1.736	-2.282	19.054	-19.054
8			3	1.736	-2.282	.014	-.014
9			4	1.736	-2.282	-19.026	19.026
10			5	1.736	-2.282	-38.067	38.067
11	2	M3	1	-2.067	2.111	-116.354	116.354
12			2	-2.067	2.111	-3.566	3.566
13			3	-.585	1.065	-52.202	52.202
14			4	-.585	1.065	4.677	-4.677
15			5	-.585	1.065	61.557	-61.557
16	2	M4	1	2.067	2.108	-116.184	116.184
17			2	2.067	2.108	-3.549	3.549
18			3	.585	1.064	-52.179	52.179
19			4	.585	1.064	4.667	-4.667
20			5	.585	1.064	61.513	-61.513

Drift Check (Sec 4.5.3.1)

Frame (seis)

Story	Story V	Col V	Drift R*	LS
R	16.9	8.45	0.028	NOT OK
2nd	16.6	8.3	0.033	NOT OK

* Per RISAs Model (EQ only combo).... Gives higher numbers than (Eqn 4-7)

Story Drift					
	Story	Joint(X)	X Drif...	% of Ht	
1	2	1	N3	4.482	3.296
2	2	2	N5	4.187	2.792

Axial Stress Check (Sec 4.5.3.6)

Frame	Col	Allow	Stress	Comply?
Grid D	(1st)	3.6	2.07	OK

Forces per RISAs model

3325 Chanate Road, Santa Rosa, CA 95404

Flexural Stress Check (Sec 4.5.3.9)

Frame Member	Grid D Top Fb	Forces per RISA model (EQ only Load combo) Bot Fb	max stress	Avg Max Stress	Fy	Comply?
M1	49.534	-49.534		80.04	ksi	37
	24.779	-24.779				NOT COMPLIANT
	0.023	-0.023	49.53			
	-24.732	24.732				
	-49.488	49.488				
M2	38.094	-38.094				
	19.054	-19.054				
	0.014	-0.014	38.09			
	-19.026	19.026				
	-38.067	38.067				
M3	-116.354	116.354				
	-3.566	3.566				
	-52.202	52.202	116.35			
	4.677	-4.677				
	61.557	-61.557				
M4	-116.184	116.184				
	-3.549	3.549				
	-52.179	52.179	116.18			
	4.667	-4.667				
	61.513	-61.513				

$$DCR_{(avg)} = \frac{80}{37} = 2.16$$

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Panel Zones (Sec A.3.1.3.5)

Beam one side of column

Columns Size	dc	bc	tcw	tcf	Beams Size	db	Zb	0.8 Mp (Beam)	Shear Demand	Shear Capacity	Comply?
W8x35	8.12	8.02	0.31	0.495	W12x16	12	20.1	578.88	48.24	64.98	Yes
W8x35	8.12	8.02	0.31	0.495	W12x26	12.2	37.2	1071.36	87.82	64.81	No

capacity equation J10-11, AISC 360

Beam both sides of column

Columns Size	dc	bc	tcw	tcf	Beams Size	db	Zb	0.8 Mp (Beam)	Shear Demand	Shear Capacity	Comply?
W8x35	8.12	8.02	0.31	0.495	W12x16	12	40.2	1157.76	96.48	64.98	No
W8x35	8.12	8.02	0.31	0.495	W12x16+W12x26	12.2	57.3	1650.24	135.27	64.81	No

capacity equation J10-11, AISC 360

Tier 2 checks

Beam both sides of column

Columns Size	dc	bc	tcw	tcf	Beams Size	db	Zb	Columns Zc	m-factor	Mp (Beam)	Shear Demand	Shear Capacity	Comply?
W8x35	8.12	8.02	0.31	0.495	W12x16	12	40.2	34.7	1.47	1591.92	132.66	128.69	No
W8x35	8.12	8.02	0.31	0.495	W12x16+W12x26	12.2	57.3	34.7	1.47	2269.08	185.99	128.69	No

Using $k=1.0$

capacity equation 9-5 ASCE 41-13

Per section 9.4.2.4.2 FR conn can be considered
Deformation controlled and evaluated using EQ 7-36
if not designed to promote yielding of the beam

$$DCR = 132/1240 = 1.03$$

$$DCR = 186/120.69 = 1.44$$

$$DCR_{avg} = 1.24$$

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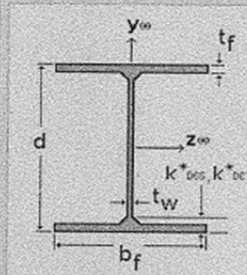
Strong Column/ Weak Beam (Sec A.3.1.3.7)**End Columns**

Level	Columns	ΣZ_c	Beams	ΣZ_b	col fa	Cpr	Ratio	Comply
Roof	W8x35	34.7	W12x16	20.1	1.04	1.2	1.40	Yes
Roof	W8x35	34.7	W12x26	37.2	1.04	1.2	0.76	No
2nd	W8x35	69.4	W12x16	20.1	2.07	1.2	2.73	Yes
2nd	W8x35	69.4	W12x26	37.2	2.07	1.2	1.48	Yes

Interior Columns

Level	Columns	ΣZ_c	Beams	ΣZ_b	col fa	Cpr	Ratio	Comply
Roof	W8x35	34.7	W12x16	40.2	1.04	1.2	0.70	No
Roof	W8x35	34.7	W12x16/26	57.3	1.04	1.2	0.49	No
2nd	W8x35	69.4	W12x16	40.2	2.07	1.2	1.37	Yes
2nd	W8x35	69.4	W12x16/26	57.3	2.07	1.2	0.96	No

Edit Shape



Shape Properties

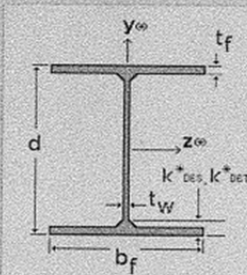
Shape Name: W8x35

Depth	8.12	in	Z _{yy}	16.1	in ³
Flange Width	8.02	in	Z _{zz}	34.7	in ³
Flange Thick	.495	in	C _w	619	in ⁶
Web Thick	.31	in	W _{no}	15.3	in ²
Area	10.3	in ²	S _w	15.2	in ⁴
I _{yy}	42.6	in ⁴	r _T	2.2	in
I _{zz}	127	in ⁴	k*des	.889	in
J	.769	in ⁴	k*det	1.19	in

k* is for Connection calcs only Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

Edit Shape



Shape Properties

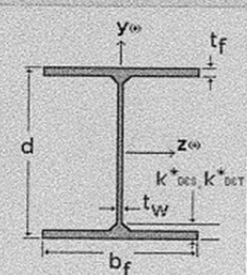
Shape Name: W12x16

Depth	12	in	Z _{yy}	2.26	in ³
Flange Width	3.99	in	Z _{zz}	20.1	in ³
Flange Thick	.265	in	C _w	96.9	in ⁶
Web Thick	.22	in	W _{no}	11.7	in ²
Area	4.71	in ²	S _w	3.09	in ⁴
I _{yy}	2.82	in ⁴	r _T	.964	in
I _{zz}	103	in ⁴	k*des	.565	in
J	.103	in ⁴	k*det	.81	in

k* is for Connection calcs only Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

Edit Shape



Shape Properties

Shape Name: W12x26

Depth	12.2	in	Z _{yy}	8.17	in ³
Flange Width	6.49	in	Z _{zz}	37.2	in ³
Flange Thick	.38	in	C _w	607	in ⁶
Web Thick	.23	in	W _{no}	19.2	in ²
Area	7.65	in ²	S _w	11.8	in ⁴
I _{yy}	17.3	in ⁴	r _T	1.72	in
I _{zz}	204	in ⁴	k*des	.68	in
J	.3	in ⁴	k*det	1.06	in

k* is for Connection calcs only Z' is the Plastic Modulus

OK Cancel Clear Data Print Help

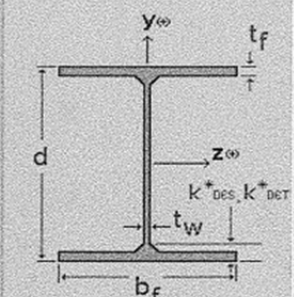
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Compactness (Sec A.3.1.3.8)

Moderately Ductile limits assumed per AISC 341-10 table D1.1

Beams	b/tf	$65/(F_y)^{0.5}$	Compact?	h/tw	$640.3/(F_y)^{0.5}$	Compact?
W12x16	7.53	10.83333333	Yes	49.40	106.7166667	Yes
W12x26	8.54	10.83333333	Yes	47.13	106.7166667	Yes
Cols	b/tf	$65/(F_y)^{0.5}$	Compact?	h/tw	$640.3/(F_y)^{0.5}$	Compact?
W8x35	8.1	10.83333333	Yes	20.5	106.7166667	Yes

Edit Shape



Shape Properties

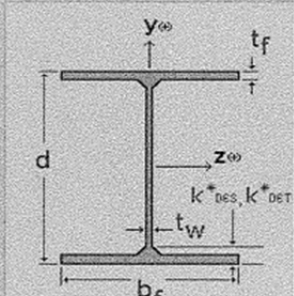
Shape Name: W8x35

Depth	8.12	in	Z _{yy}	16.1	in ³
Flange Width	8.02	in	Z _{zz}	34.7	in ³
Flange Thick	.495	in	C _w	619	in ⁶
Web Thick	.31	in	W _{no}	15.3	in ²
Area	10.3	in ²	S _w	15.2	in ⁴
I _{yy}	42.6	in ⁴	r _T	2.2	in
I _{zz}	127	in ⁴	k*des	.889	in
J	.789	in ⁴	k*det	1.19	in

k* is for Connection calcs only Z is the Plastic Modulus

OK Cancel Clear Data Print Help

Edit Shape



Shape Properties

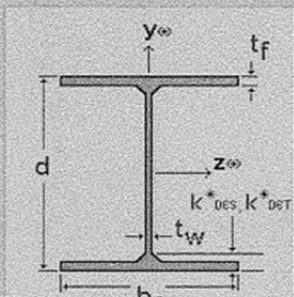
Shape Name: W12x16

Depth	12	in	Z _{yy}	2.26	in ³
Flange Width	3.99	in	Z _{zz}	20.1	in ³
Flange Thick	.265	in	C _w	96.9	in ⁶
Web Thick	.22	in	W _{no}	11.7	in ²
Area	4.71	in ²	S _w	3.09	in ⁴
I _{yy}	2.82	in ⁴	r _T	.964	in
I _{zz}	103	in ⁴	k*des	.565	in
J	.103	in ⁴	k*det	.81	in

k* is for Connection calcs only Z is the Plastic Modulus

OK Cancel Clear Data Print Help

Edit Shape



Shape Properties

Shape Name: W12x26

Depth	12.2	in	Z _{yy}	8.17	in ³
Flange Width	6.49	in	Z _{zz}	37.2	in ³
Flange Thick	.38	in	C _w	607	in ⁶
Web Thick	.23	in	W _{no}	19.2	in ²
Area	7.65	in ²	S _w	11.8	in ⁴
I _{yy}	17.3	in ⁴	r _T	1.72	in
I _{zz}	204	in ⁴	k*des	.68	in
J	.3	in ⁴	k*det	1.06	in

k* is for Connection calcs only Z is the Plastic Modulus

OK Cancel Clear Data Print Help

3325 Chanate Road, Santa Rosa, CA 95404

BLDG 3 - AREA B STEEL ADDITION WEIGHTS

Floors	Weight (psf)	Areas	Misc Area	DL (KIP)
Roof	22	318	0	7
2nd	60	318	0	19

Roof Walls	Weight (psf)	Height	Length	DL (KIP)
LINE 1	15	8.33	17	2.1
LINE 2	15	8.33	17	2.1
LINE A	15	8.33	0	0.0
LINE B	15	8.33	0	0.0
LINE C	15	8.33	19	2.4
TOTAL =				6.6

2nd Floor Walls	Weight (psf)	Height	Length	DL (KIP)
LINE 1	15	11.33	17	2.9
LINE 2	15	11.33	17	2.9
LINE A	15	11.33	0	0.0
LINE B	15	11.33	0	0.0
LINE C	15	11.33	19	3.2
TOTAL =				9.0

Columns	Weight (plf)	Height	Quantity	DL (KIP)
Roof	35	5.66	4	1
2nd	35	11.33	4	2

Grand Total Dead Loads Per Floor	DL (KIP)	Floor Area	KSF/FLR
Roof	14	427	0.034
2nd	30	427	0.069
Total	44k		
V =	1.097W		

Seismic Story Force Distribution based on ASCE 41-13

$$T_a \text{ Period (7.4.1.2.2)} = 0.425 \quad k = 1 \quad (7.4.1.3.2)$$

$$V(ULT) = 1.097$$

$$\text{Base } V(ULT) = 48,268$$

Story Force Vertical Distribution (ASCE 41-13 7.4.1.3.2)

Level	w_x	h_x (ft.)	h_x^k	$w_x h_x^k$	F_x , ULT	Load to frame	$C_v\%$
ROOF	14,000	23.7	23.7	331380	23,828	11,914	49.4
2nd	30,000	11.3	11.3	339900	24,440	12,220	50.6
Σ	44,000			671280	48,268		

Vertical Diaphragm Distribution (ASCE 41-13 7.4.1.3.4)

Level	w_x	Σw_x	F_x	ΣF_x	F_{px} , ULT
ROOF	14,000	14,000	23,828	23,828	23,828
2nd	30,000	44,000	24,440	48,268	32,910
Σ	44,000		48,268		

3325 Chanate Road, Santa Rosa, CA 95404

ZFA STRUCTURAL ENGINEERS

Building 3 – Area B Addition

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ROOF PLAN VIEW

DF

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section

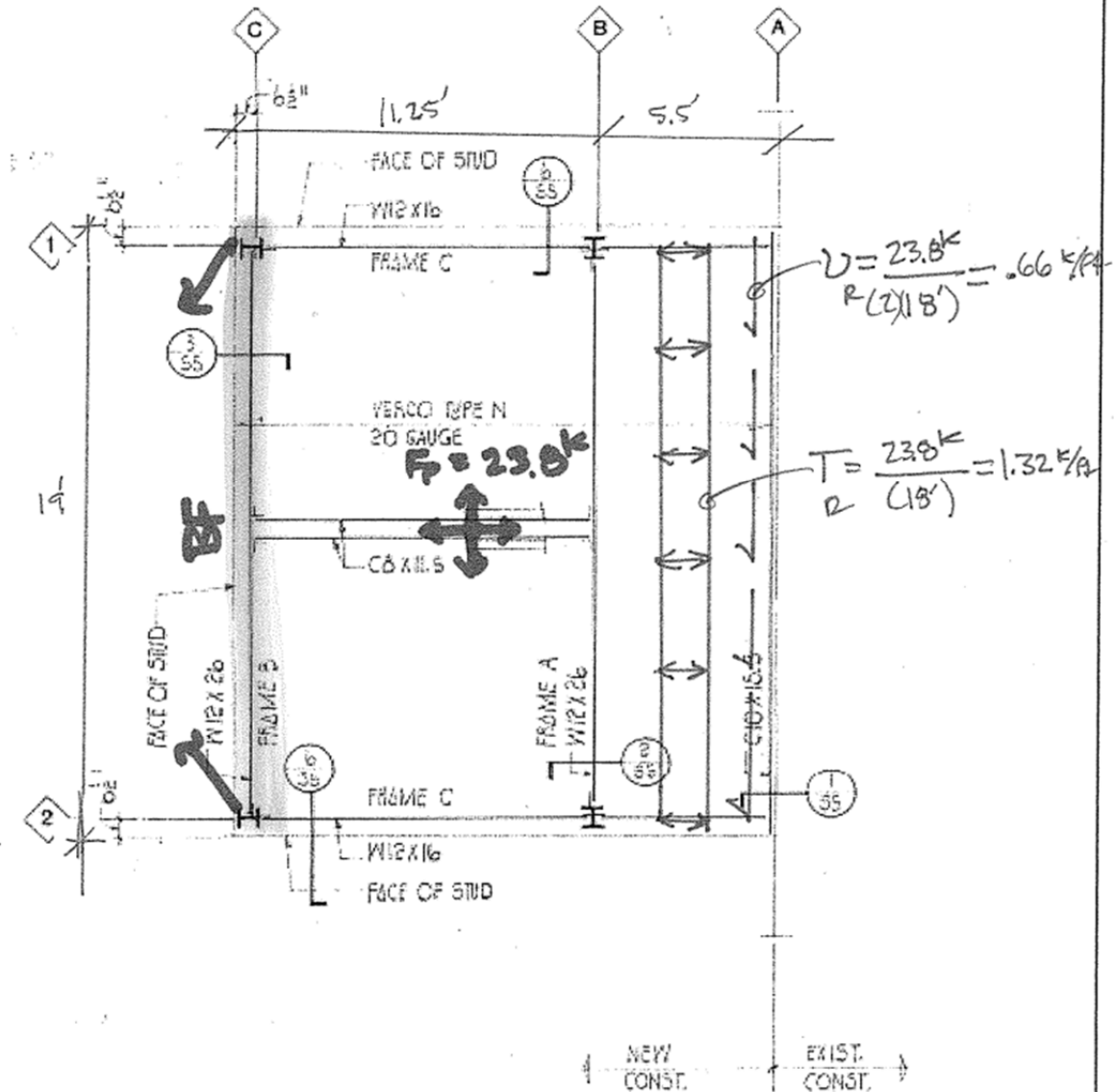
engineer

job #

date

page

- CHECK LEDGER CONNECTION AT AREA B (60's)



$A_{TOT} = 310 SF$
RF

3325 Chanate Road, Santa Rosa, CA 95404

ZFA STRUCTURAL ENGINEERS

Building 3 – Area B Addition

project name

FLOOR PLAN VIEW

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section

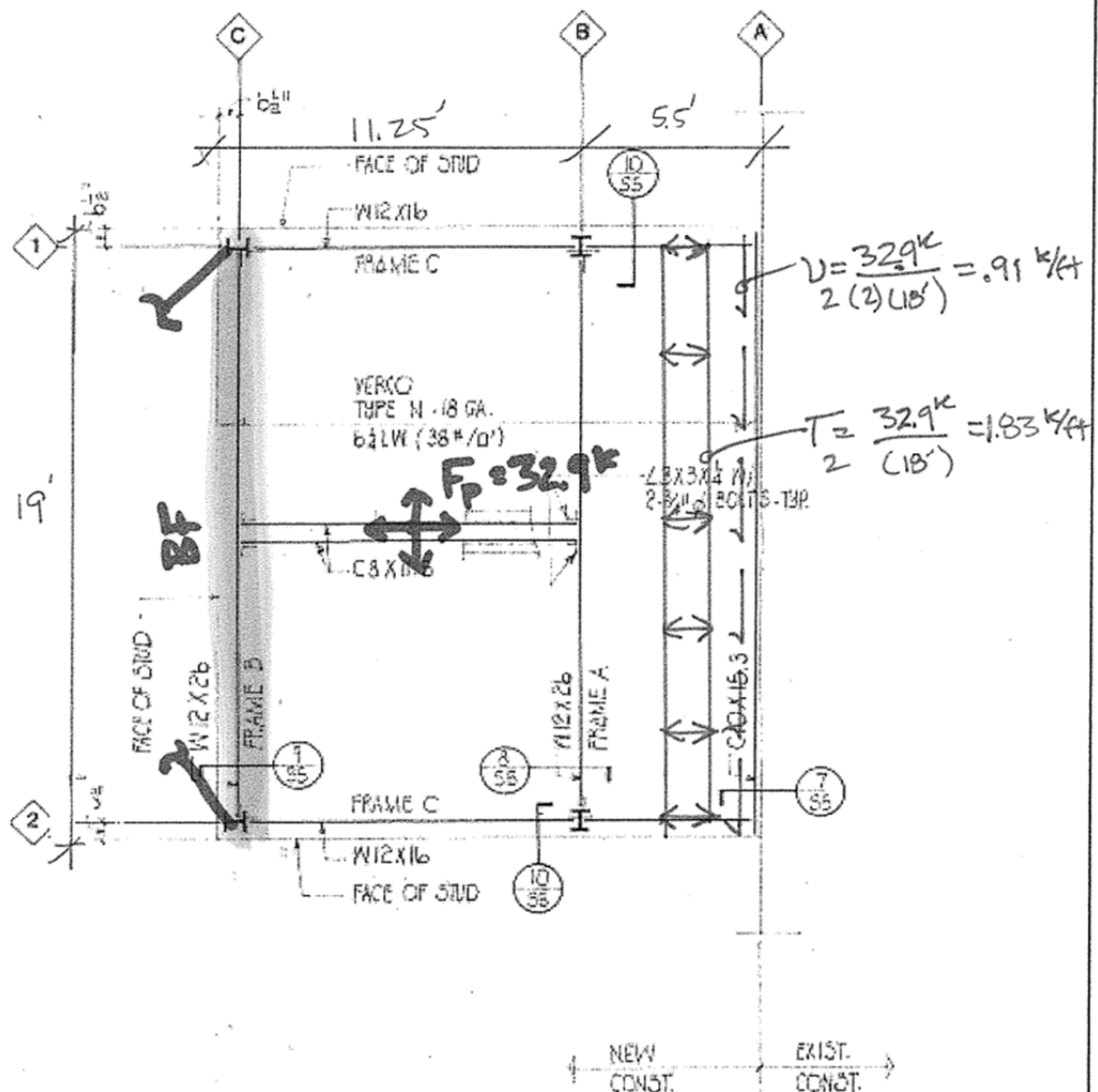
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job #

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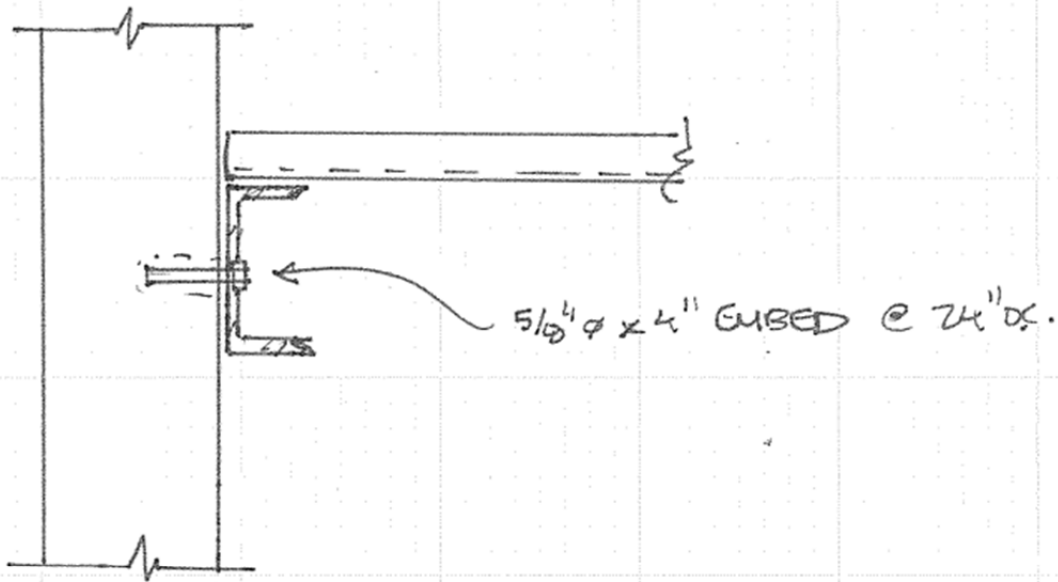
page

- CHECK LEDGER CONNECTION



$A_{TOT} = 319 SF$
 F_{LR}

3325 Chanate Road, Santa Rosa, CA 95404

LEDGER CONNECTION:

$$V_{QU} = 7865 \#^* \times 12" / 24" OC = 3.934 \text{ K/ft} \quad (* \text{SEE OUTPUT})$$

$$T_{QU} = 1360 \#^* \times 12" / 24" OC = .68 \text{ K/ft} \quad (* \text{SEE OUTPUT})$$

$$RQ_u \geq Q_u + \frac{Q_E}{C_1 C_2 J}$$

$$C_1 C_2 = 1.1$$

$$J = 1.0 \quad (\text{ABs TO REMAIN ELASTIC})$$

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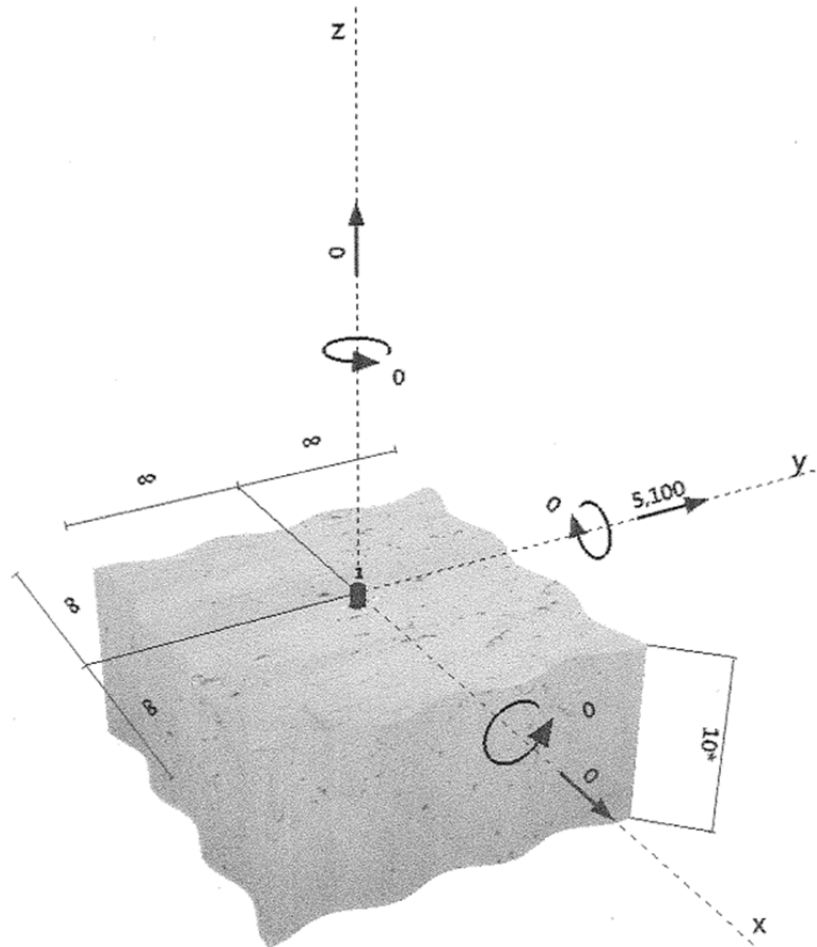
Specifier's comments:

1 Input data

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 5/8
Effective embedment depth: $h_{ef} = 4.000$ in.
Material: ASTM F 1554
Proof: design method ACI 318-08 / CIP
Stand-off installation: - (Recommended plate thickness: not calculated)
Profile: no profile
Base material: cracked concrete, 2500, $f_c' = 2500$ psi; $h = 10.000$ in.
Reinforcement: tension: condition B, shear: condition B;
 edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F) yes (D.3.3.4)



Geometry [in.] & Loading [lb, in.lb]



Input data and results must be checked for agreement with the existing conditions and for plausibility!
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4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	5100	5112	100	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	5100	10080	51	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading ** anchor group (relevant anchors)

4.1 Steel Strength

$$V_{sa} = n \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} \quad \text{ACI 318-08 Eq. (D-20)}$$

$$\phi V_{steel} \geq V_{ua} \quad \text{ACI 318-08 Eq. (D-2)}$$

Variables

n	$A_{se,V}$ [in. ²]	f_{uta} [psi]
1	0.23	58000

Calculations

$$V_{sa} \text{ [lb]} = 7865$$

Results

V_{sa} [lb]	ϕ_{steel}	ϕV_{sa} [lb]	V_{ua} [lb]
7865	0.650	5112	5100

$V_{OCL} = 7865^*$

4.2 Pryout Strength

$$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nco}} \right) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right] \quad \text{ACI 318-08 Eq. (D-30)}$$

$$\phi V_{cp} \geq V_{ua} \quad \text{ACI 318-08 Eq. (D-2)}$$

$$A_{Nc} \text{ see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)}$$

$$A_{Nco} = 9 h_{ef}^2 \quad \text{ACI 318-08 Eq. (D-6)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-9)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-11)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-13)}$$

$$N_b = k_c \lambda \sqrt{f_c} h_{ef}^{1.5} \quad \text{ACI 318-08 Eq. (D-7)}$$

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	4.000	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ	f_c [psi]
1.000	-	24	1	2500

Calculations

A_{Nc} [in. ²]	A_{Nco} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
144.00	144.00	1.000	1.000	1.000	1.000	9600

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	ϕV_{cp} [lb]	V_{ua} [lb]
19200	0.700	0.750	10080	5100

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1 Input data

A 3D perspective diagram showing a rectangular plate of dimensions 12 by 10 resting on a wavy surface. A coordinate system is defined with the z -axis pointing vertically upwards, the y -axis pointing along the length of the plate, and the x -axis pointing along the width of the plate. The origin is at the center of the plate. Dimensions are labeled: the plate's length is 10, its width is 12, and its thickness is 0.25. The z -axis has a vertical arrow pointing up with a value of 1.360. The y -axis has a curved arrow indicating a rotation. The x -axis has a curved arrow indicating a rotation. The wavy surface is labeled with a value of 10. The z -axis is labeled with a value of 8. The y -axis is labeled with a value of 8. The x -axis is labeled with a value of 8.

ZFA STRUCTURAL ENGINEERS

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2 Load case/Resulting anchor forces

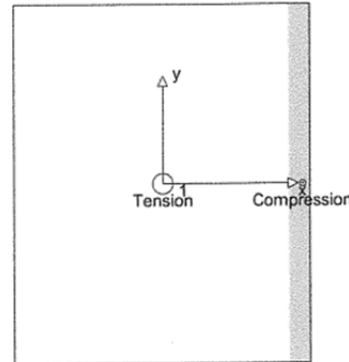
Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2718	0	0	0

max. concrete compressive strain: 0.07 [%]
 max. concrete compressive stress: 309 [psi]
 resulting tension force in (x/y)=(0.000/0.000): 2718 [lb]
 resulting compression force in (x/y)=(4.756/0.000): 1358 [lb]



3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	2718	9831	28	OK
Pullout Strength*	2718	1907	143	not recommended
Concrete Breakout Strength**	2718	2016	135	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$$N_{sa} = n A_{se,N} f_{uta} \quad \text{ACI 318-08 Eq. (D-3)}$$

$$\phi N_{steel} \geq N_{ua} \quad \text{ACI 318-08 Eq. (D-1)}$$

Variables

n	$A_{se,N}$ [in. ²]	f_{uta} [psi]
1	0.23	58000

Calculations

$$N_{sa} \text{ [lb]} = 13108$$

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
13108	0.750	9831	2718

3.2 Pullout Strength

$$N_{pN} = \psi_{c,p} N_p \quad \text{ACI 318-08 Eq. (D-14)}$$

$$N_p = 8 A_{brg} f_c \quad \text{ACI 318-08 Eq. (D-15)}$$

$$\phi N_{pN} \geq N_{ua} \quad \text{ACI 318-08 Eq. (D-1)}$$

Variables

$\psi_{c,p}$	A_{brg} [in. ²]	f_c [psi]
1.000	0.45	2500

Calculations

$$N_p \text{ [lb]} = 9080$$

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{pn} [lb]	N_{ua} [lb]
9080	0.700	0.750	0.400	1907	2718

$$Q_u^* = 1907 \times \frac{1}{.7} = 2724^*$$

$$T_{ou} = 1360^* (\text{e 4.75" C.L.T.})$$

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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3.3 Concrete Breakout Strength

$$N_{cb} = \left(\frac{A_{Nc}}{A_{Nco}} \right) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-08 Eq. (D-4)}$$

$$\phi N_{cb} \geq N_{ua} \quad \text{ACI 318-08 Eq. (D-1)}$$

 A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$$A_{Nco} = 9 h_{ef}^2 \quad \text{ACI 318-08 Eq. (D-6)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-9)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-11)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-13)}$$

$$N_b = k_c \lambda \sqrt{f'_c} h_{ef}^{1.5} \quad \text{ACI 318-08 Eq. (D-7)}$$

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
4.000	0.000	0.000	∞	1.000
c_{ac} [in.]	k_c	λ	f'_c [psi]	
0.000	24	1	2500	

Calculations

A_{Nc} [in. ²]	A_{Nco} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
144.00	144.00	1.000	1.000	1.000	1.000	9600

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{cb} [lb]	N_{ua} [lb]
9600	0.700	0.750	0.400	2016	2718

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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SHEAR:

ROOF:

$$(.75)(3.934) \geq \frac{.66}{(1.1)(1.0)}$$

$$2.95 \text{ K/ft} \geq .6 \text{ K/ft} \quad \underline{\underline{\text{OK}}}$$

2nd FLR:

$$(.75)(3.934) \geq \frac{.91}{(1.1)(1.0)}$$

$$2.95 \text{ K/ft} \geq .83 \text{ K/ft} \quad \underline{\underline{\text{OK}}}$$

TENSION:

ROOF:

$$(.75)(.68) \geq \frac{1.32}{(1.1)(1.0)}$$

$$DCR = 1.2 / .51 = 2.35$$

$$.51 \text{ K/ft} \geq 1.2 \text{ K/ft} \quad \underline{\underline{\text{N.G.}}}$$

→ DBL SPCG OF A, B'S

2nd FLR:

$$(.75)(.68) \leq \frac{1.63}{(1.1)(1.0)}$$

$$DCR = 1.66 / .51 = 3.25$$

$$.51 \text{ K/ft} \leq 1.66 \text{ K/ft} \quad \underline{\underline{\text{N.G.}}}$$

→ TRIPLE SPCG OF A, B'S

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Specifier's comments:

1 Input data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 3/4 (6 1/4)

Effective embedment depth: $h_{ef} = 4.840$ in., $h_{nom} = 6.250$ in.

Material: Carbon Steel

Evaluation Service Report: ESR-3027

Issued | Valid: 3/1/2014 | 12/1/2015

Proof: design method ACI 318 / AC193

Stand-off installation: $e_b = 0.000$ in. (no stand-off); $t = 0.250$ in.

Anchor plate: $l_x \times l_y \times t = 10.000$ in. \times 12.000 in. \times 0.250 in.; (Recommended plate thickness: not calculated)

Profile: no profile

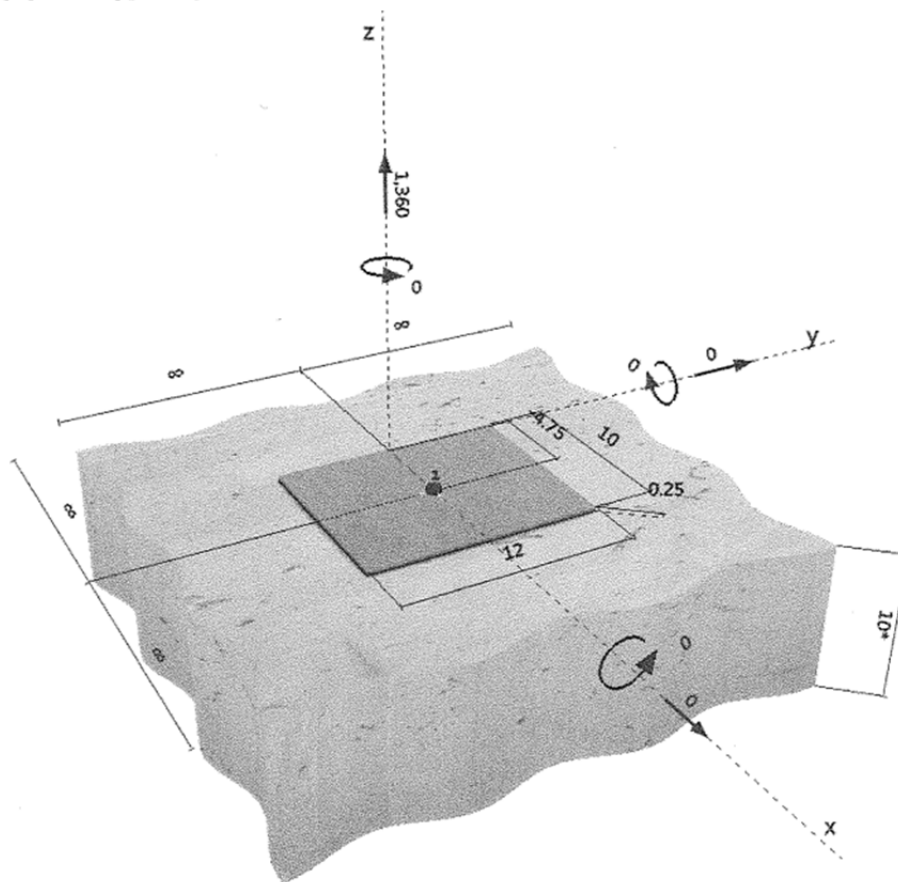
Base material: cracked concrete, 2500, $f'_c = 2500$ psi; $h = 10.000$ in.

Reinforcement: tension: condition B, shear: condition B; no supplemental splitting reinforcement present
 edge reinforcement: none or \leq No. 4 bar
 yes (D.3.3.6)

Seismic loads (cat. C, D, E, or F) yes (D.3.3.6)



Geometry [in.] & Loading [lb, in.lb]



Input data and results must be checked for agreement with the existing conditions and for plausibility!
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2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2739	0	0	0

max. concrete compressive strain:

0.06 [%]

max. concrete compressive stress:

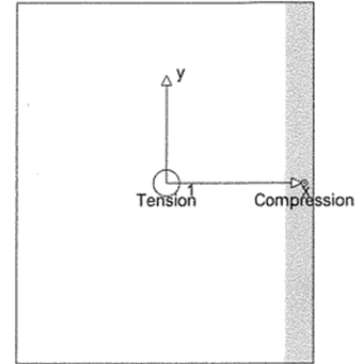
244 [psi]

resulting tension force in (x/y)=(0.000/0.000):

2739 [lb]

resulting compression force in (x/y)=(4.686/0.000):

1379 [lb]



3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	2739	8323	33	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	2739	1765	156	not recommended

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

 N_{sa} = ESR value

refer to ICC-ES ESR-3027

 $\phi N_{steel} \geq N_{ua}$

ACI 318-08 Eq. (D-1)

Variables

n	$A_{se,N}$ [in. ²]	f_{uta} [psi]
1	0.39	81600

Calculations

N_{sa} [lb]
32013

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	ϕN_{sa} [lb]	N_{ua} [lb]
32013	0.650	0.400	8323	2739

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3.2 Concrete Breakout Strength

$$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-08 Eq. (D-4)}$$

$$\phi N_{cb} \geq N_{ua} \quad \text{ACI 318-08 Eq. (D-1)}$$

 A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-08 Eq. (D-6)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-9)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-11)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-08 Eq. (D-13)}$$

$$N_b = k_c \lambda \sqrt{f'_c} h_{ef}^{1.5} \quad \text{ACI 318-08 Eq. (D-7)}$$

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
4.840	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f'_c [psi]
7.280	17	1	2500

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
210.83	210.83	1.000	1.000	1.000	1.000	9051

Results

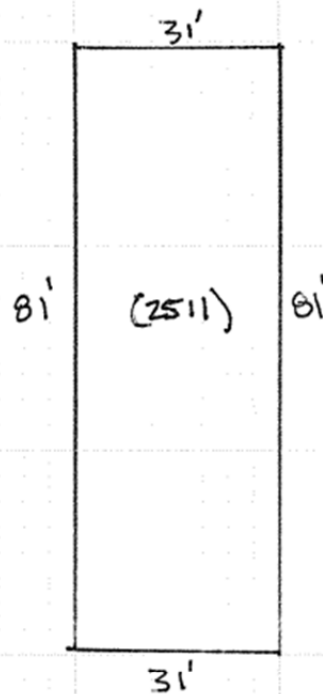
N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{ductile}$	ϕN_{cb} [lb]	N_{ua} [lb]
9051	0.650	0.750	0.400	1765	2739

$$Q_u^* = 1765 \times \frac{1}{.65} = 2716^{\#}$$

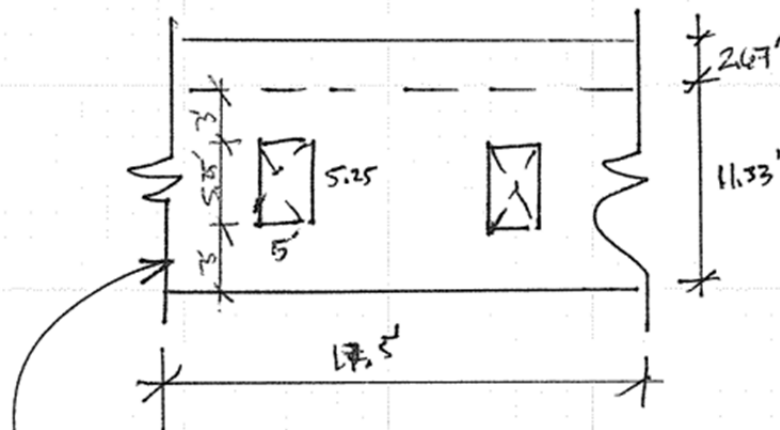
≈ EQUIVALENT TO (E)
5/8" ϕ A.B.

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Building 4 (1956 Hospital Wing)

1956 - EAST

$$A_{FLR} = 2511 \text{ SF}$$

WALL ELEVATION

$$\text{Full HT WALLS} = 11.33' / 2 + 2.67' = 8.33'$$

$$\text{WALLS w/ OPENINGS} = 3' + 2.67' = 5.67'$$

BSE-1E

$$S_s = .995$$

$$F_a = 1.002 \rightarrow S_{x_s} = .997$$

$$S_i = .389$$

$$F_v = 1.41 \rightarrow S_{x_i} = .548$$

$$T = C_t h_n^{\beta}$$

$$= .02 (11.33')^{.75} = .124$$

$$S_a = \frac{S_{x_i}}{T} = \frac{.548}{.124} = 4.42 > S_{x_s} \leftarrow \text{USE } S_{x_s}$$

$$S_a = .997$$

$$C_m = 1.0 \quad (1\text{-STORY})$$

$$m_{max} = 2.13 \quad (\text{SEE FOLLOWING CALCS})$$

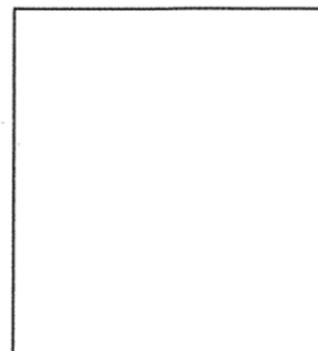
$$C_1 C_2 = 1.4$$

$$C_1 C_2 C_m = 1.4$$

$$V = (1.4) (.997) W$$

$$V = 1.395 W$$

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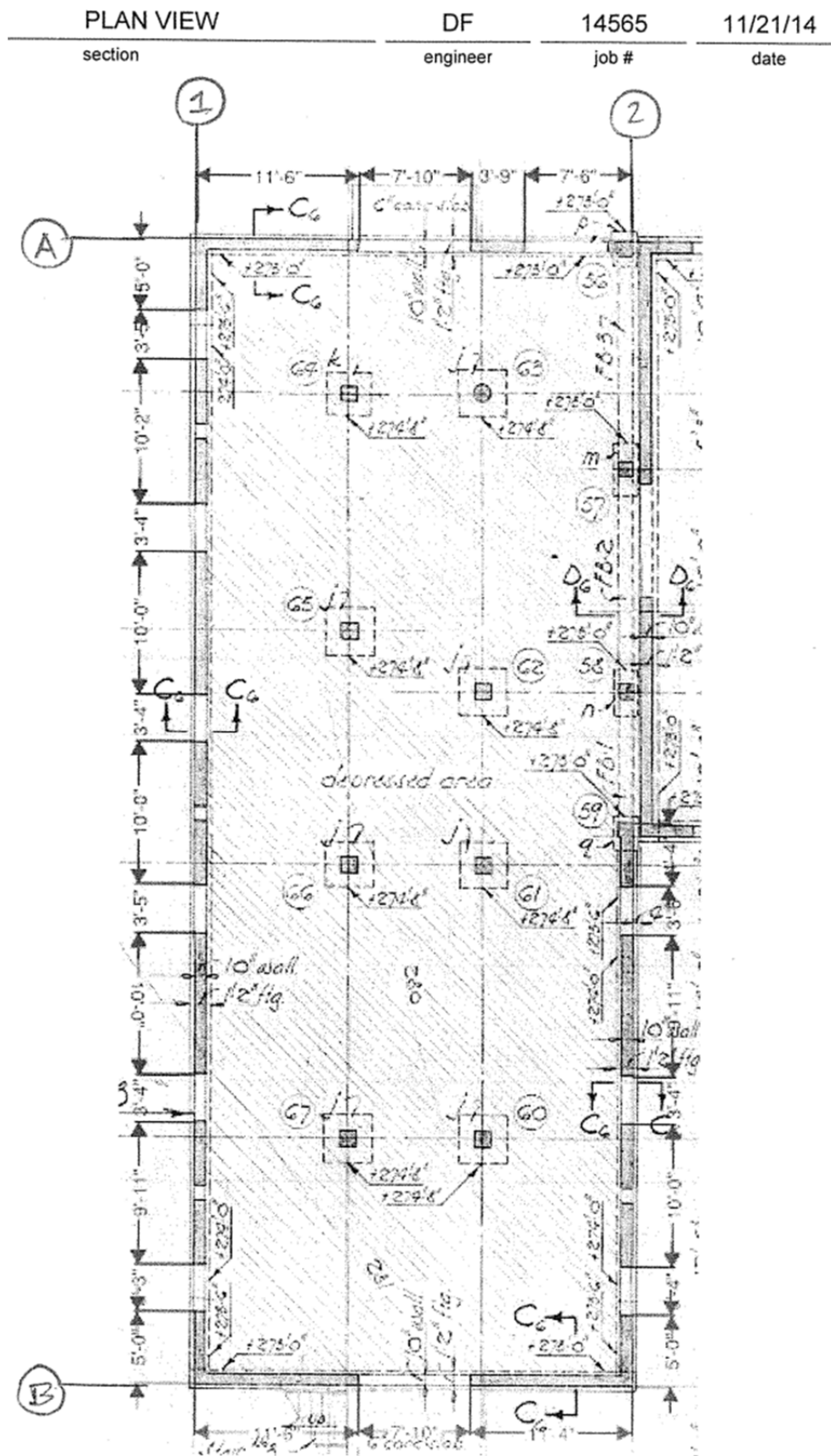
DESIGN CRITERIA**Material (unless noted otherwise)**Concrete: $f_c = 2500$ psi (per existing plans)Reinf. Steel: $f_y = 40000$ psi (per existing plans)

Stamp

DESIGN LOADING

			ROOF	FLOOR	INTERIOR WALLS	EXTERIOR WALLS
LIVE LOADS (PSF)			20.0			
DEAD LOADS (PSF)						
Roofing**	Light Weight Fill		20.0			
Fin. Floor			0.0			
Diaphragm	4.5" Conc Slab		56.3			
Joists/Truss			0.0			
Beams	In weight calcs		0.0			
Ceiling	Plaster Ceiling		8.0			
Insulation			0.0			
HVAC			2.0			
Partitions			0.0			
Sprinklers			1.5			
Misc.			2.2			
DEAD LOADS (PSF)	0.0	0.0	90.0	0.0	0.0	125.0
TOTAL LOADS (PSF)	0.0	0.0	110.0	0.0		

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LINE 1:

$$\Sigma \text{ Full-HT walls} = 5 + 10 + 10 + 10 + 10 + 10 + 5 = 60'$$

$$\Sigma \text{ walls w/ openings} = 81' - 60' = 21'$$

LINE 2:

$$\Sigma \text{ Full-HT walls} = 5 + 10 + 10 + 4.5' = 29.5'$$

$$\Sigma \text{ walls w/ openings} = 40.5' - 29.5' = 11'$$

LINE A:

$$\Sigma \text{ Full-HT walls} = 11.5' + 3.75' = 15.25'$$

$$\Sigma \text{ walls w/ openings} = 31' - 15.25' = 15.75'$$

LINE B:

$$\Sigma \text{ Full-HT walls} = 11.5' + 11.5' = 23'$$

$$\Sigma \text{ walls w/ openings} = 31' - 23' = 8'$$

$$\Sigma \text{ Full HT walls} = 60' + 29.5' + 15.25' + 23' = 128'$$

$$\Sigma \text{ walls w/ openings} = 21' + 11' + 15.75' + 8' = 56'$$

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BLDG 4 - 1956 EAST WEIGHTS

Floors	Weight (psf)	Areas	Misc Area	DL (KIP)		
2nd	90	2511	0	226.0		
Full Ht Walls	Weight (psf)	Height	Length	DL (KIP)		
LINE 1	125	8.33	60	62.5		
LINE 2	125	8.33	29.5	30.7		
LINE A	125	8.33	15.25	15.9		
LINE B	125	8.33	23	23.9		
Walls w/ Opngs	Weight (psf)	Height	Length	DL (KIP)		
LINE 1	125	5.67	21	14.9		
LINE 2	125	5.67	11	7.8		
LINE A	125	5.67	15.75	11.2		
LINE B	125	5.67	8	5.7		
Columns	Weight (psf)	Height	Quantity	DL (KIP)		
2nd	267	5.67	11	16.7		
Beams	Weight (psf)	Length	Quantity	DL (KIP)		
2nd	233	81	2	37.7		
Grand Total Dead Loads Per Floor				DL (KIP)	Floor Area	KSF/FLR
2nd				453	2511	0.180
V =				1.395W		
Vertical Distribution of Seismic Forces	Height	Weight	wi*hi	wi*hi/sum(wihi)	FX	Force per floor (kips)
2nd	11.33	453	5132	1.00	632	632
		453	5132	1		632

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CENTER OF MASS

Building Overall Dimensions	X=	31.0 ft.
	Y=	81.0 ft.

ITEM	WEIGHT	cmx	cmy	W*cmx	W*cmy
ROOF SLAB	226.0	15.5	40.5	3503	9153
WALL LINE 1	77.4	0.0	40.5	0	3135
WALL LINE 2	38.5	31.0	40.5	1194	1559
WALL LINE A	27.0	15.5	81.0	419	2187
WALL LINE B	30.0	15.5	0.0	465	0
				0	0
				0	0
				0	0
				0	0
				0	0
				0	0
	399			5580	16034

CMX=	14.0 ft.
------	----------

CMY=	40.2 ft.
------	----------

Accidental torsion, 5%	ex=	1.6	ey=	4.1
Amplification Factor, Ax	Ax =	1.0	Ax =	1.0
Amplified eccentricity	Amp ex=	1.6	Amp ey=	4.1

CMX + Amp ex=	15.5 ft.
CMX - Amp ex=	12.4 ft.

CMY + Amp ey=	44.2 ft.
CMY - Amp ey=	36.1 ft.

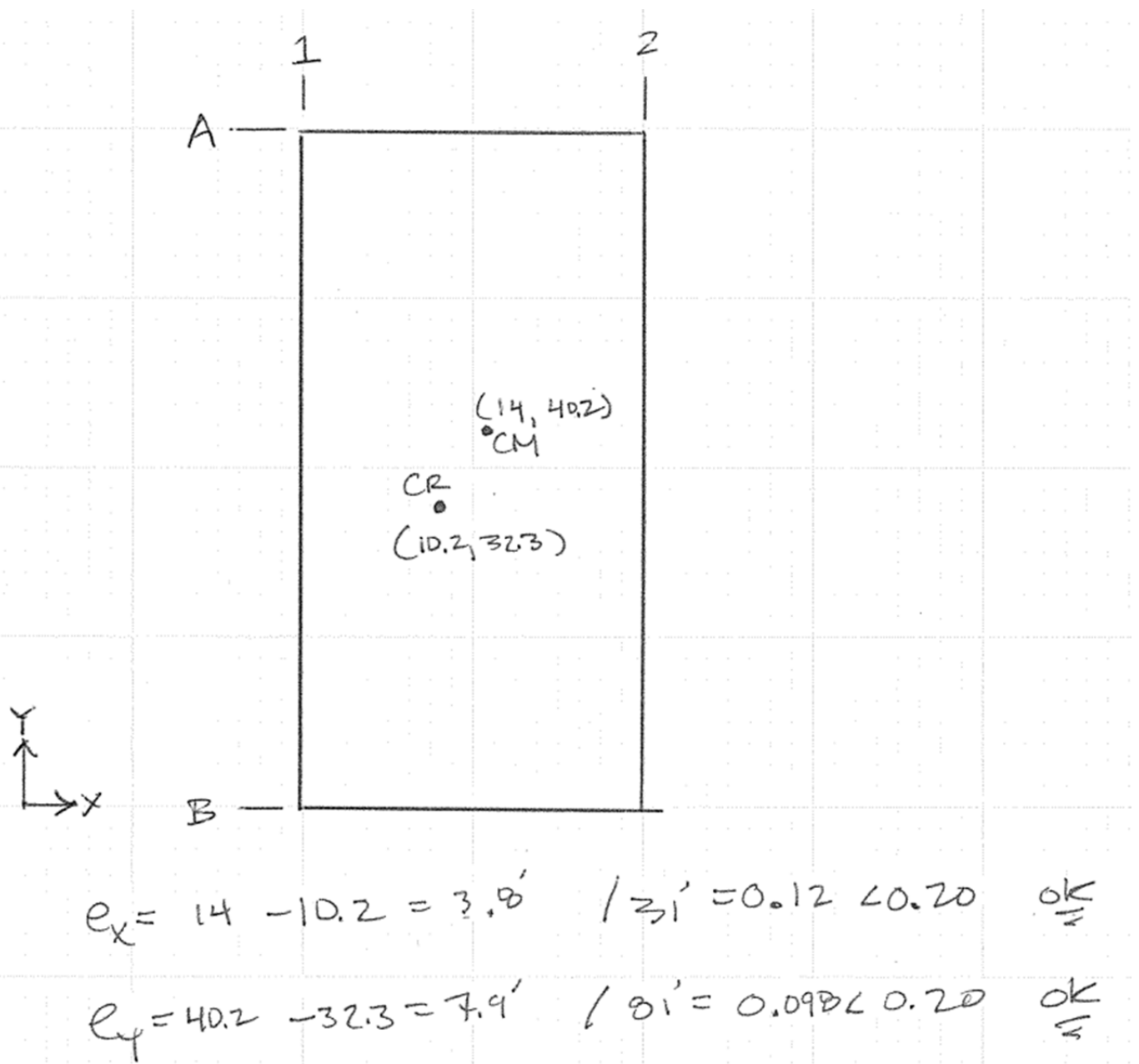
CENTER OF RIGIDITY**CRY (WALLS IN THE X DIRECTION)**

WALL	Description	R_x	Y	R_x*Y
A		15.25	81.00	1235.25
B		23.00	0.00	0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
SUM		38.25		1235.25
CRY=				32.3 ft.

CRX (WALLS IN THE Y DIRECTION)

WALL	Description	R_y	X	R_y*X
1		60.00	0.00	0.00
2		29.50	31.00	914.50
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
SUM		89.50		914.50
CRX=				10.2 ft.

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HORIZONTAL DISTRIBUTION

FORCE DIRECTION =	X
V =	632 kips

CMY + Ey -12 ft.
M = V*e = -7553 kip-ft

CMY - Ey -4 ft.
M = V*e = -2434 kip-ft

WALL	Rx		Y	D	R*D	R*D^2	Fv	Fm @ + Ey	Fm @ - Ey	Ftotal
WALLS IN THE X DIRECTION										Max
A	15.3		81.0	-48.7	-742.8	36177.0	252.0	70.9	22.8	322.8
B	23.0		0.0	32.3	742.8	23986.9	380.0	-70.9	-22.8	357.2
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Rx= 38.3

60163.9

← GOV

WALL		Ry	X	D	R*D	R*D^2	Fv	Fm @ + Ey	Fm @ - Ey	Ftotal
WALLS IN THE Y DIRECTION										max
1		60.0	0.0	10.2	613.1	6264.3	0.0	-58.5	-18.8	-58.5
2		29.5	31.0	-20.8	-613.1	12741.0	0.0	58.5	18.8	58.5
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Ry= 89.5

19005.3

SUM OF RD^2= 79169.2

→ WALL LINE A CARRIES "SECONDARY FRAME" AND ALSO HAS MAX EQL LOAD (K/FT)

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HORIZONTAL DISTRIBUTION

FORCE DIRECTION =	Y
V =	632 kips

CMX + Ex -5 ft.

M = V*e = -3363 kip-ft

CMX - Ex -2 ft.

M = V*e = -1403 kip-ft

WALL	Rx		Y	D	R*D	R*D^2	Fv	Fm @ + Ex	Fm @ - Ex	Ftotal
WALLS IN THE X DIRECTION										Max
A	15.3		81.0	-48.7	-742.8	36177.0	0.0	31.5	13.2	31.5
B	23.0		0.0	32.3	742.8	23986.9	0.0	-31.5	-13.2	-31.5
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	32.3	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Rx= 38.3

60163.9

WALL		Ry	X	D	R*D	R*D^2	Fv	Fm @ + Ex	Fm @ - Ex	Ftotal
WALLS IN THE Y DIRECTION										Max
1		60.0	0.0	10.2	613.1	6264.3	423.7	-26.0	-10.9	412.8
2		29.5	31.0	-20.8	-613.1	12741.0	208.3	26.0	10.9	234.4
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	10.2	0.0	0.0	0.0	0.0	0.0	0.0

SUM OF Ry=

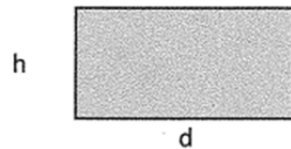
89.5

19005.3

SUM OF RD^2= 79169.2

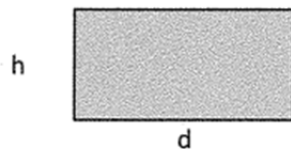
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Determine load distribution based on rigidity of panels:

WALL LINE: A (Gov's in X-Dir by Inspection)LINE LOAD: 322.8 kPanel 1

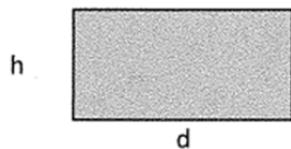
h = 15.67 ft
d = 11.5 ft

$$\begin{aligned} h/d &= 1.36 \\ \Delta &= 0.4(h/d)^3 + 0.3(h/d) = 1.421 \\ R &= 1/\Delta = 0.704 \end{aligned}$$

Panel 2

h = 15.67 ft
d = 3.75 ft

$$\begin{aligned} h/d &= 4.18 \\ \Delta &= 0.4(h/d)^3 + 0.3(h/d) = 30.440 \\ R &= 1/\Delta = 0.033 \end{aligned}$$

Panel 3

h = 15.67 ft
d = 1.5 ft

$$\begin{aligned} h/d &= 10.45 \\ \Delta &= 0.4(h/d)^3 + 0.3(h/d) = 459.164 \\ R &= 1/\Delta = 0.002 \end{aligned}$$

Distribution of Shear to Each Panel

Panel	Rigidity	Shear %	Shear Force (k)	Shear Force (k/l)
1	0.704	0.95	307.50	26.74
2	0.033	0.04	14.35	3.83
3	0.002	0.00	0.95	0.63
Σ	0.739	1.000	322.8	31.2

(Max loaded SW in Bldg)

SW CHECK

section

DF

engineer

14565

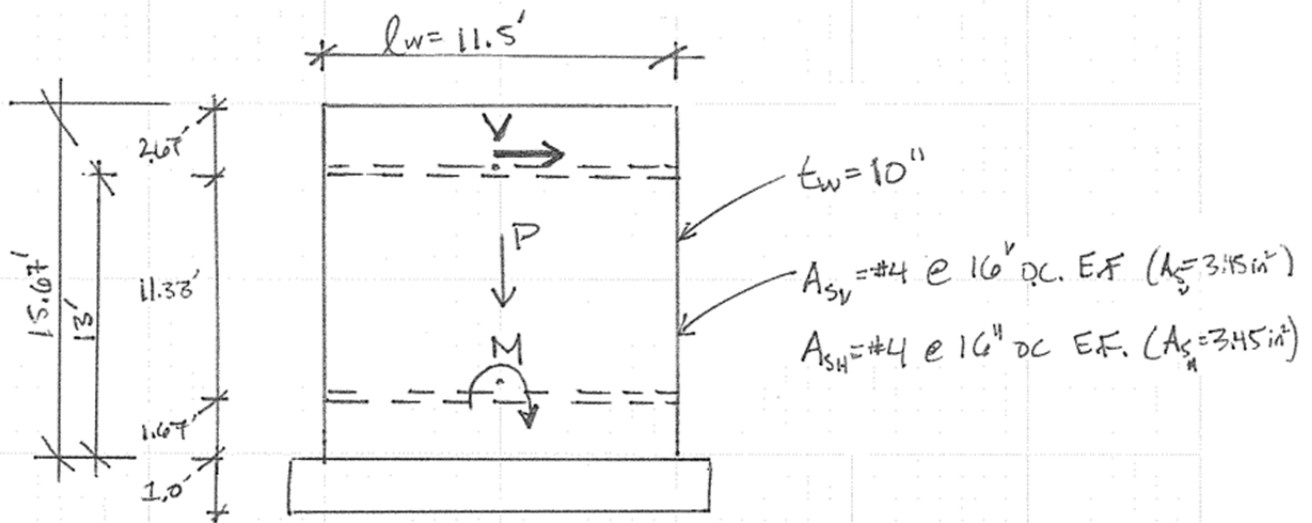
job #

11/14

date

page

SW ELEVATION



$$P = (90 \text{ PSF})(10.75'/2)(11.5') + (10''/12 \times 150 \text{ PSF})(15.67')(11.5') = 28.1^k$$

$$V = 300^k \text{ (QCE)}$$

$$M = (300^k)(13') - (28.1^k)(11.5'/2) = 3843^k\text{-ft} \text{ (QCE)}$$

DESIGN CRITERIA FOR SHEAR WALLS

$$K = 0.9$$

$$f'_c = 2500 \text{ psi}$$

$$f'_{ce} = 2500 \times 1.5 = 3750 \text{ psi} \quad \leftarrow \text{TABLE 10-1}$$

$$f_y = 40,000 \text{ psi}$$

$$f_{ye} = 40,000 \times 1.25 = 50,000 \text{ psi} \quad \leftarrow \text{TABLE 10-1}$$

TABLE 10-21

$$\frac{(A_s - A'_s) f_y + P}{t_w l_w f'_c} = \frac{(.5 \times 3.45)(40) + (29.1)}{(10)(11.5 \times 12)(25)} = .03 \leq 0.1$$

$$\frac{V}{t_w l_w \sqrt{f'_c}} = \frac{300 \text{ K} \times 1000}{(10)(11.5 \times 12)(\sqrt{2500})} = 4.5 > 4 < 6$$

$$\frac{2.5 - 2}{6 - 4} = \frac{m - 2}{4.5 - 4}$$

$$m = 2.13$$

[CONTROLLED BY
FLEXURE]

CHECK SHEAR IN WALL

$$Q_{CE} = 2 \sqrt{f'_c} h d + \frac{(A_{sw})(f_y)(d)}{(s)}$$

$$d = .8 l_w \text{ (ACI 11.9.4)}$$

$$d = .8 (11.5' \times 12) = 110.4''$$

$$Q_{CE} = 2(\sqrt{3750})(10'')(110.4'') + \frac{(.40)(50 \text{ ksi})(110.4'')}{(16'')}$$

$$Q_{CE} = 135^k + 138^k = 273^k$$

$$M K Q_{CE} = (2.13)(1.9)(273^k) = 523^k > 300^k \quad \underline{\underline{OK}}$$

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CHECK FLEXURE IN WALL

- ASSUME TENSION CONTROLLED (NOT COMPRESSION CONTROLLED)

$$\omega = \frac{A_s f_y}{f'_c l_w t} = \frac{(3.45)(50)}{(3.75)(11.5 \times 12)(10)} = .033$$

$$\alpha = \frac{P_u}{f'_c l_w t} = \frac{(28.1)}{(3.75)(11.5 \times 12)(10)} = .0057$$

$$\frac{c}{l_w} = \frac{\omega + \alpha}{2\omega + \alpha \beta_1} = \frac{.033 + .0057}{2(.033) + (.85 \times .85)} = .049$$

$$M_n = .5 A_s f_y l_w \left(1 + \frac{P}{A_s f_y}\right) \left(1 - \frac{c}{l_w}\right)$$

$$M_n = (.5)(3.45)(50)(11.5 \times 12) \left(1 + \frac{28.1}{(3.45 \times 50)}\right) (1 - .049)$$

$$M_n = 13,262 \text{ k-ft}$$

$$mKQ_{CE} = (2.13)(1.9)(13,262) = 25,423 \text{ k-ft} > 3034 \text{ k-ft}$$

OK

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MISC CALLS:SHEAR WALLS:

$$V_j^{AVG} = \frac{1}{M_s} \left(\frac{V_j}{A_w} \right) \rightarrow \text{CHECK WORST CASE DIR AT BASE}$$

$$A_w = (10\frac{11}{12}) (31' + 31') (1.5) = 25.8 \text{ SF}$$

↑ CORNERS

$$v = \left(\frac{1}{4.0} \right) \left(\frac{773^k \times 1000}{25.8 \times 144 / \text{ft}^2} \right) = 52 \text{ psi} < 100 \text{ psi} \quad \underline{\text{OK}}$$

WALL REINF:

#4 @ 16" o.c. EW EF (TYP)

$$A_s = .2 \times 12" / 16" \text{ o.c.} \times 2 \text{ FACES} = .30 \text{ in}^2 / \text{ft}$$

$$\rho = \frac{.30 \text{ in}^2}{10" \times 12} = .0025 > .0020 \text{ (H)}$$

$$> .0012 \text{ (V)}$$

OKCOUPLING BEAMS:

$$d_{\text{TYP}} = 5'-3" = 66"$$

$$d/2 = 66"/2 = 33" > 16" \text{ o.c.}$$

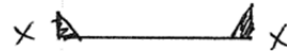
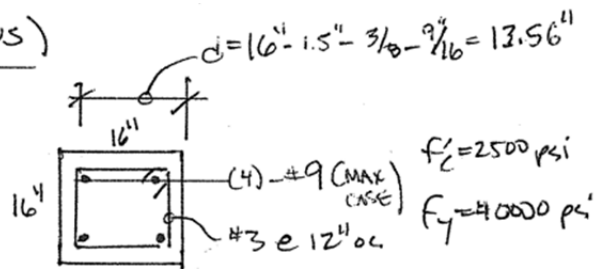
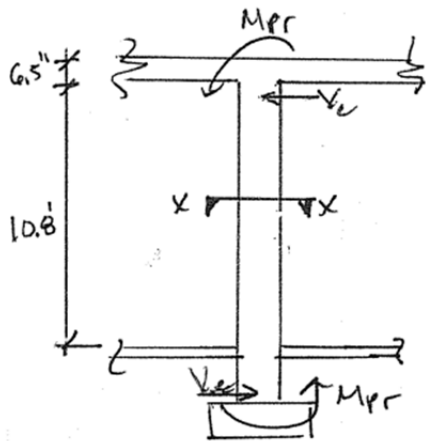
SPC6

 ← NO 135° HOOKS ON STIRRUPS
ADJACENT BLOBS:

$$11.33' \times .04 = 5.4" < 8"$$

OKOK

3325 Chanate Road, Santa Rosa, CA 95404

DEFLECTION COMPATABILITY (COLUMNS)

$$V_e = \frac{M_{pr} - (-M_{pr})}{L} \leq V_n$$

$$M_{pr} = 1.25 A_s f_y (d - a/2)$$

$$a = \frac{(2 \times 1.0)(40,000)}{(0.85)(2500)(16)} = 2.35$$

$$M_{pr} = 1.25 (2 \times 1.0)(40)(13.63 - 2.35/2) = 1246 \text{ k-in}$$

$$\frac{M_{pr} - (-M_{pr})}{L} = \frac{1246 \text{ k-in} \times 2}{10.8' \times 12} = 19 \text{ k}$$

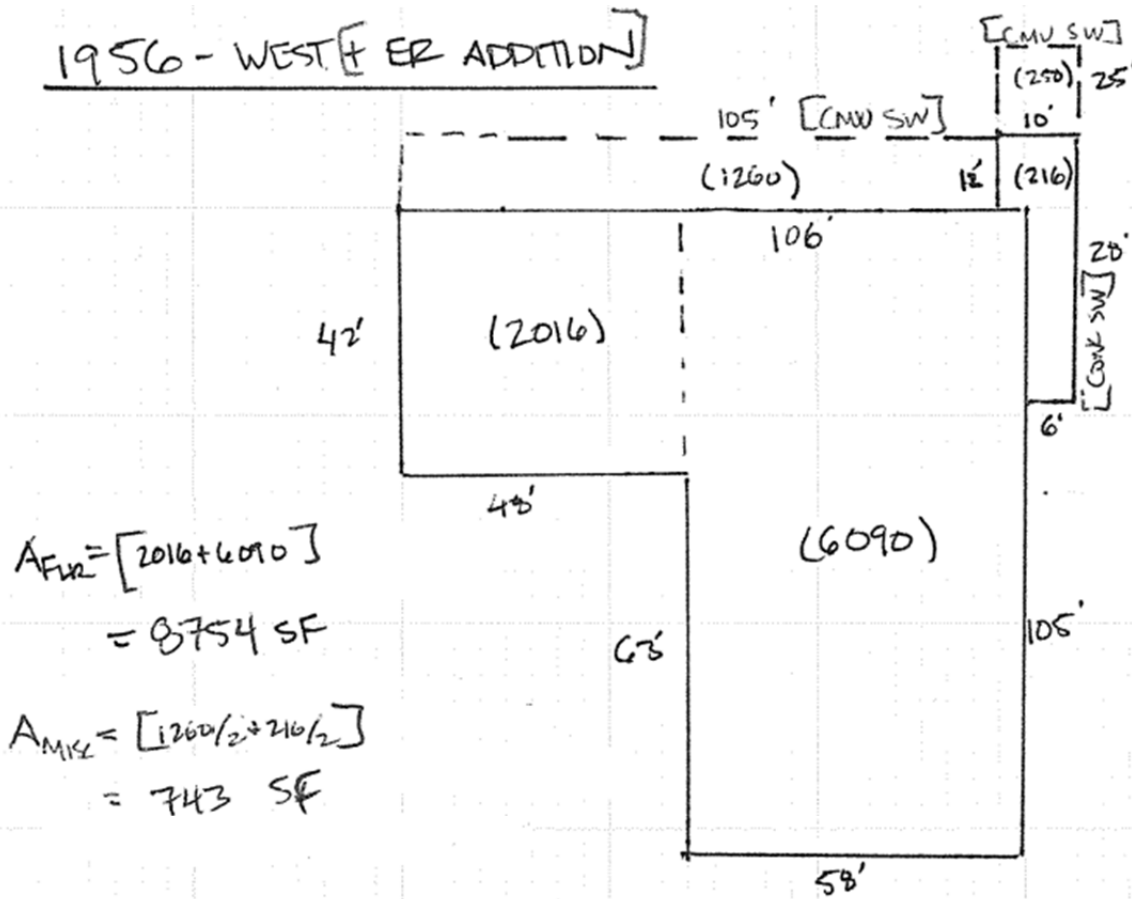
$$V_n = V_s + V_c \rightarrow 0 \text{ (NEGLECT CONC DURING EQ EVENT)}$$

$$V_s = \frac{A_v f_y d}{s} = \frac{(0.11 \text{ in}^2 \times 2)(40 \text{ ksi})(13.56'')}{4''} = 29.9 \text{ k} > V_e$$

4" SPCG IN TOP 12" OF COLUMN

OK

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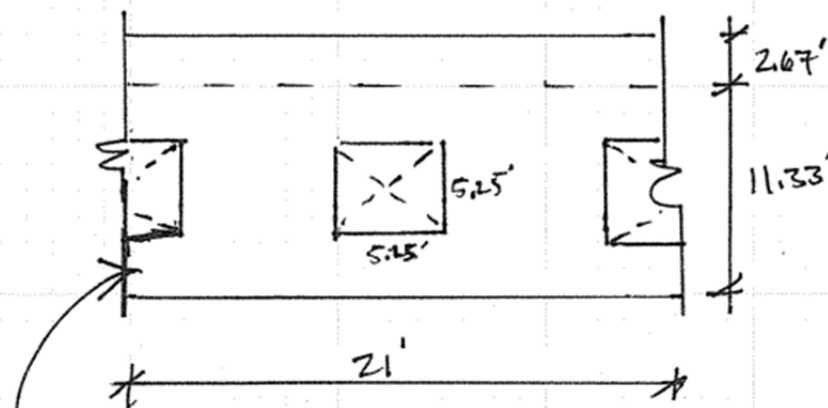
Building 5 (1956 Hospital Wing)1956 - WEST [ER ADDITION]

$$A_{FLR} = [2016 + 6090]$$

$$= 8754 \text{ SF}$$

$$A_{MISC} = [1260/2 + 216/2]$$

$$= 743 \text{ SF}$$



$$\text{Full HT WALLS} = 11.33/2 + 2.67' = 8.33'$$

$$\text{WALLS w/ ORINGS} = 3' + 2.67' = 5.67'$$

BSE-1E

$$S_s = .995$$

$$F_a = 1.002 \rightarrow S_{x_s} = .997$$

$$S_i = .389$$

$$F_v = 1.410 \rightarrow S_{x_i} = .548$$

$$T = C_t h_n^{1/3}$$

$$= .02 (11.33')^{1/3} = .124$$

$$S_a = \frac{S_{x_i}}{T} = \frac{.548}{.124} = 4.42 > S_{x_s} \leftarrow \text{USE } S_{x_s}$$

$$S_a = .997$$

$$C_m = 1.0 \text{ (1-STORY)}$$

$$m_{max} = 2.43$$

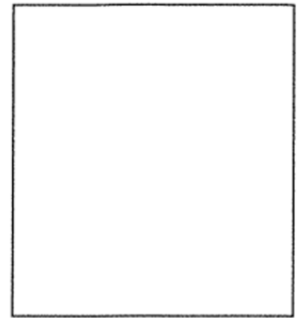
$$C_1 C_2 = 1.4$$

$$C_1 C_2 C_M = 1.4$$

$$V = (1.4)(1.397)W$$

$$V = 1.395 W$$

3325 Chanate Road, Santa Rosa, CA 95404

DESIGN CRITERIA**Material (unless noted otherwise)**Concrete: $f'_c = 2500$ psi (per existing plans)Reinf. Steel: $f_y = 40000$ psi (per existing plans)

Stamp

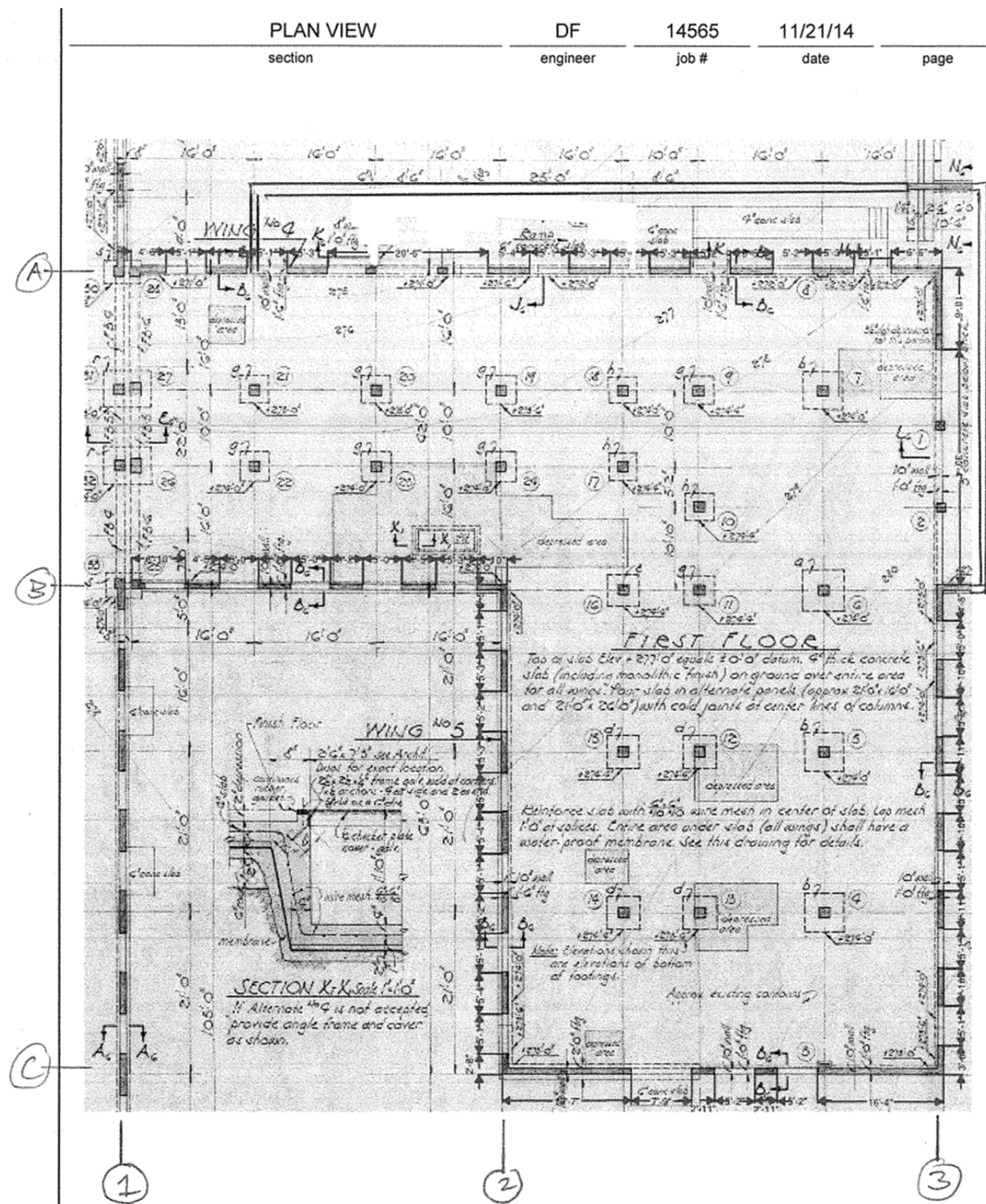
DESIGN LOADING

			ROOF	FLOOR	INTERIOR WALLS	EXTERIOR WALLS
LIVE LOADS (PSF)			20.0			
DEAD LOADS (PSF)						
Roofing**	Light Weight Fill		20.0			
Fin. Floor			0.0			
Diaphragm	6.5" Conc Slab		81.3			
Joists/Truss			0.0			
Beams	14"x18" Average		9.0			
Ceiling			2.0			
Insulation			0.0			
HVAC			2.0			
Partitions			0.0			
Sprinklers			1.5			
Misc.			2.2			
DEAD LOADS (PSF)	0.0	0.0	118.0	0.0	0.0	125.0
TOTAL LOADS (PSF)	0.0	0.0	138.0	0.0		

Geotechnical Report by:

 Rutherford & Chekene
 2002-112G
 20-Dec-02

3325 Chanate Road, Santa Rosa, CA 95404



LINE 1:

$$\sum \text{FULL-HT WALLS} = 0$$

$$\sum \text{WALL w/ OPNGS} = 0$$

LINE 2:

$$\sum \text{FULL-HT WALLS} = 3.5' + 5(5.25') + 2.67' = 32.5'$$

$$\sum \text{WALL w/ OPNGS} = 68' - 32.5' = 30.5'$$

LINE 3:

$$\sum \text{FULL-HT WALLS} = 10.5' + 6(4.83') + 3.67' = 42.25'$$

$$\sum \text{WALL w/ OPNGS} = 105' - 42.25' = 62.75'$$

LINE A:

$$\sum \text{FULL-HT WALLS} = (4.5' \times 4) + 4' = 22'$$

$$\sum \text{WALL w/ OPNGS} = 48' - 22' = 26'$$

LINE B:

$$\sum \text{FULL-HT WALLS} = 4.5' + (5.25' \times 7) + 6.5' = 48'$$

$$\sum \text{WALLS w/ OPNGS} = 106' - 48' = 58'$$

LINE C:

$$\sum \text{FULL-HT WALLS} = 16.5' + 3' + 3' + 16.5' = 39'$$

$$\sum \text{WALLS w/ OPNGS} = 58' - 39' = 19'$$

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BLDG 5 - 1956 WEST WEIGHTS

Floors	Weight (psf)	Areas	Misc Area	DL (KIP)		
2nd	115	8106	648	1007		
Full Ht Walls	Weight (psf)	Height	Length	DL (KIP)		
LINE 1	125	8.33	0	0.0		
LINE 2	125	8.33	32.5	33.8		
LINE 3	125	8.33	42.25	44.0		
LINE A	125	8.33	22	22.9		
LINE B	125	8.33	48	50.0		
LINE C	125	8.33	39	40.6		
Walls w/ Opngs	Weight (psf)	Height	Length	DL (KIP)		
LINE 1	125	5.67	0	0.0		
LINE 2	125	5.67	30.5	21.6		
LINE 3	125	5.67	62.75	44.5		
LINE A	125	5.67	26	18.4		
LINE B	125	5.67	58	41.1		
LINE C	125	5.67	19	13.5		
Columns	Weight (psf)	Height	Quantity	DL (KIP)		
2nd	267	5.67	22	33.3		
Beams	Weight (psf)	Length	Quantity	DL (KIP)		
2nd	233	153	2	71.3		
Grand Total Dead Loads Per Floor				DL (KIP)	Floor Area	KSF/FLR
				2nd	1442	8754
						0.165
				V =	1.395W	
Vertical Distribution of Seismic Forces	Height	Weight	wi*hi	wi*hi/sum(wihi)	FX	Force per floor (kips)
2nd	11.33	1442	16335	1.00	2011	2011
		1442	16335	1		2011

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CENTER OF MASS

Building Overall Dimensions	X=	106.0 ft.
	Y=	105.0 ft.

ITEM	WEIGHT	cmx	cmy	W*cmx	W*cmy
ROOF SLAB A	232	24	84	5568	19488
ROOF SLAB B	700	77	53	53900	36750
ER ADDITION ROOF SLAB	75	61	110	4575	8250
WALL LINE 1	0.0	0.0	84.0	0	0
WALL LINE 2	60.1	48.0	31.5	2885	1893
WALL LINE 3	88.5	106.0	52.5	9381	4646
WALL LINE A	41.3	53.0	105.0	2189	4337
WALL LINE B	91.1	24.0	63.0	2186	5739
WALL LINE C	54.1	77.0	0.0	4166	0
				0	0
				0	0
	1342			84850	81103

CMX= 63.2 ft.

CMY= 60.4 ft.

Accidental torsion, 5% ex= 5.3

ey= 5.3

Amplification Factor, Ax = 1.0

Ax = 1.0

Amplified eccentricity Amp ex= 5.3

Amp ey= 5.3

CMX + Amp ex= 68.5 ft.

CMX - Amp ex= 57.9 ft.

CMY + Amp ey= 65.7 ft.

CMY - Amp ey= 55.2 ft.

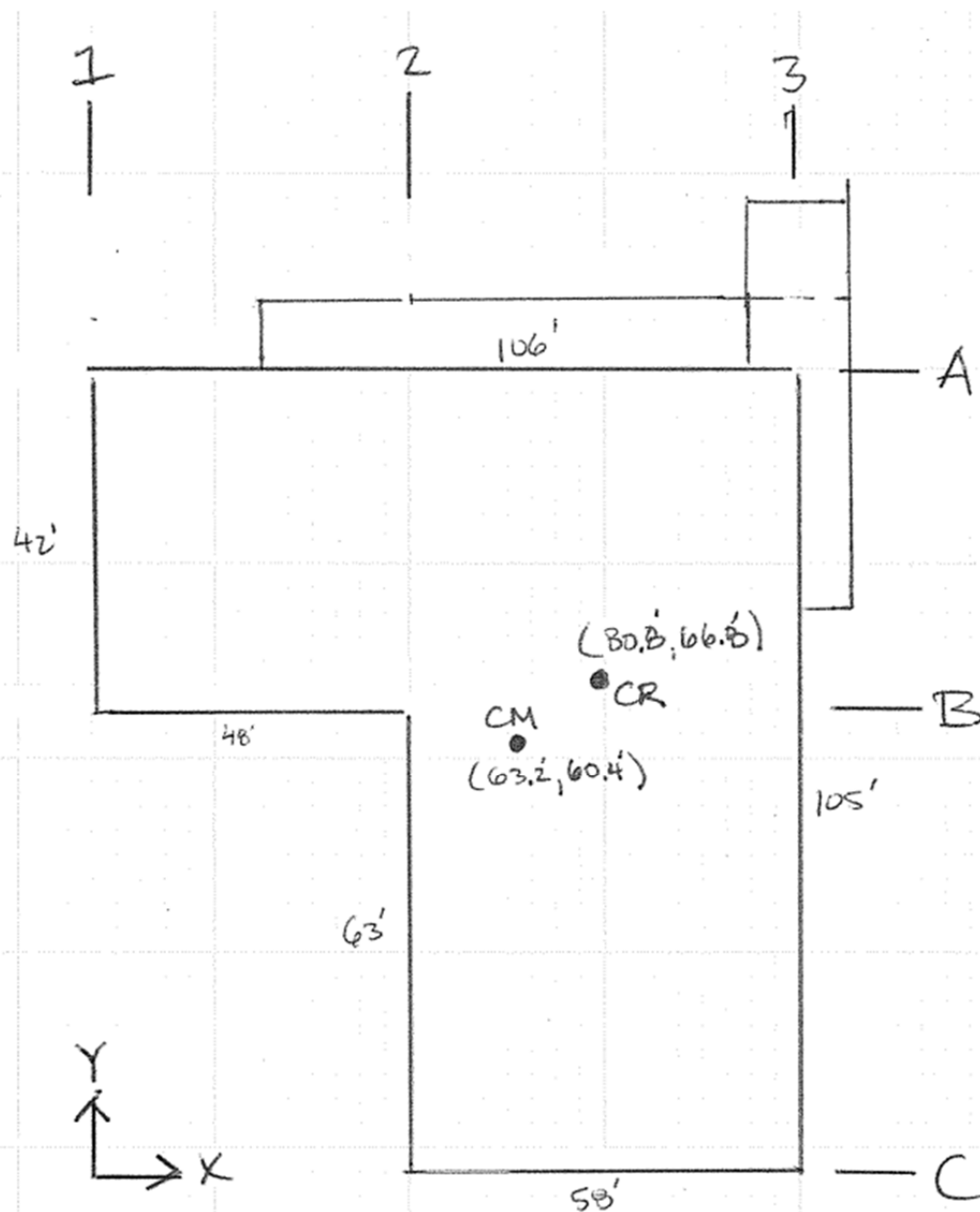
CENTER OF RIGIDITY**CRY (WALLS IN THE X DIRECTION)**

WALL	Description	R _x	Y	R _x *Y
A		106.00	105.00	11130.00
B		48.00	63.00	3024.00
C		58.00	0.00	0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
SUM		212.00		14154.00
			CRY=	66.8 ft.

CRX (WALLS IN THE Y DIRECTION)

WALL	Description	R _y	X	R _y *X
1		0.00	0.00	0.00
2		32.50	48.00	1560.00
3		42.25	106.00	4478.50
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
				0.00
SUM		74.75		6038.50
			CRX=	80.8 ft.

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$$e_x = 80.8' - 63.2' = 17.6' / 106' = .17 < 0.20 \quad \text{OK}$$

$$e_y = 66.8' - 60.4' = 6.4' / 105' = .06 < 0.20 \quad \text{OK}$$

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HORIZONTAL DISTRIBUTION

FORCE DIRECTION =	Y
V =	2011 kips

CMX + Ex 12 ft.

M = V*e = 24657 kip-ft

CMX - Ex 23 ft.

M = V*e = 45973 kip-ft

WALL	Rx		Y	D	R*D	R*D^2	Fv	Fm @ + Ex	Fm @ - Ex	Ftotal
WALLS IN THE X DIRECTION										Max
A	106.0		105.0	-38.2	-4053.0	154969.9	0.0	-210.0	-391.5	-391.5
B	48.0		63.0	3.8	180.7	680.1	0.0	9.4	17.5	17.5
C	58.0		0.0	66.8	3872.3	258532.2	0.0	200.6	374.0	374.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
SUM OF Rx= 212.0										414182.2

WALL		Ry	X	D	R*D	R*D^2	Fv	Fm @ + Ex	Fm @ - Ex	Ftotal
WALLS IN THE Y DIRECTION										Max
1		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
2		32.5	48.0	32.8	1065.4	34927.7	874.3	55.2	102.9	977.3
3		42.3	106.0	-25.2	-1065.4	26867.5	1136.7	-55.2	-102.9	1081.5
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
SUM OF Ry= 74.8										61795.2

SUM OF RD^2= 475977.4

→ WALL LINE 2 CARRIES "SECONDARY
FRAMES" AND ALSO HAS MAX EG LOAD (K/FT)

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HORIZONTAL DISTRIBUTION

FORCE DIRECTION =	X
V =	2011 kips

CMY + Ey 1 ft.
M = V*e = 2180 kip-ft

CMY - Ey 12 ft.
M = V*e = 23296 kip-ft

WALL	Rx		Y	D	R*D	R*D^2	Fv	Fm @ + Ey	Fm @ - Ey	Ftotal
WALLS IN THE X DIRECTION										Max
A	106.0		105.0	-38.2	-4053.0	154969.9	1005.5	-18.6	-198.4	986.9
B	48.0		63.0	3.8	180.7	680.1	455.3	0.8	8.8	464.2
C	58.0		0.0	66.8	3872.3	258532.2	550.2	17.7	189.5	739.7
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0		0.0	66.8	0.0	0.0	0.0	0.0	0.0	0.0
SUM OF Rx= 212.0										414182.2

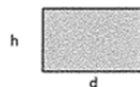
WALL		Ry	X	D	R*D	R*D^2	Fv	Fm @ + Ey	Fm @ - Ey	Ftotal
WALLS IN THE Y DIRECTION										max
1		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
2		32.5	48.0	32.8	1065.4	34927.7	0.0	4.9	52.1	52.1
3		42.3	106.0	-25.2	-1065.4	26867.5	0.0	-4.9	-52.1	-52.1
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
0		0.0	0.0	80.8	0.0	0.0	0.0	0.0	0.0	0.0
SUM OF Ry= 74.8										61795.2
										SUM OF RD^2= 475977.4

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Determine load distribution based on rigidity of panels:

WALL LINE: 2
 LINE LOAD: 977.3 k

Panel 1



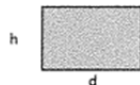
h = 14 ft
 d = 3.5 ft

$$h/d = 4.00$$

$$\Delta = 0.4(h/d)^3 + 0.3(h/d) = 26.800$$

$$R = 1/\Delta = 0.037$$

Panel 2



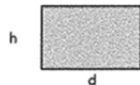
h = 14 ft
 d = 5.25 ft

$$h/d = 2.67$$

$$\Delta = 0.4(h/d)^3 + 0.3(h/d) = 8.385$$

$$R = 1/\Delta = 0.119$$

Panel 3



h = 14 ft
 d = 5.25 ft

$$h/d = 2.67$$

$$\Delta = 0.4(h/d)^3 + 0.3(h/d) = 8.385$$

$$R = 1/\Delta = 0.119$$

Panel 4



h = 14 ft
 d = 5.25 ft

$$h/d = 2.67$$

$$\Delta = 0.4(h/d)^3 + 0.3(h/d) = 8.385$$

$$R = 1/\Delta = 0.119$$

Panel 5



h = 14 ft
 d = 5.25 ft

$$h/d = 2.67$$

$$\Delta = 0.4(h/d)^3 + 0.3(h/d) = 8.385$$

$$R = 1/\Delta = 0.119$$

Panel 6



h = 14 ft
 d = 5.25 ft

$$h/d = 2.67$$

$$\Delta = 0.4(h/d)^3 + 0.3(h/d) = 8.385$$

$$R = 1/\Delta = 0.119$$

Panel 7



h = 14 ft
 d = 2.67 ft

$$h/d = 5.24$$

$$\Delta = 0.4(h/d)^3 + 0.3(h/d) = 59.238$$

$$R = 1/\Delta = 0.017$$

Distribution of Shear to Each Panel

Panel	Rigidity	Shear %	Shear Force (k)	Shear Force (k/l)
1	0.037	0.06	56.06	16.02
2	0.119	0.18	179.18	51.19
3	0.119	0.18	179.18	51.19
4	0.119	0.18	179.18	51.19
5	0.119	0.18	179.18	51.19
6	0.119	0.18	179.18	51.19
7	0.017	0.03	25.36	7.25
Σ	0.650	1.000	977.3	279.2

SW CHECK

section

DF

engineer

14565

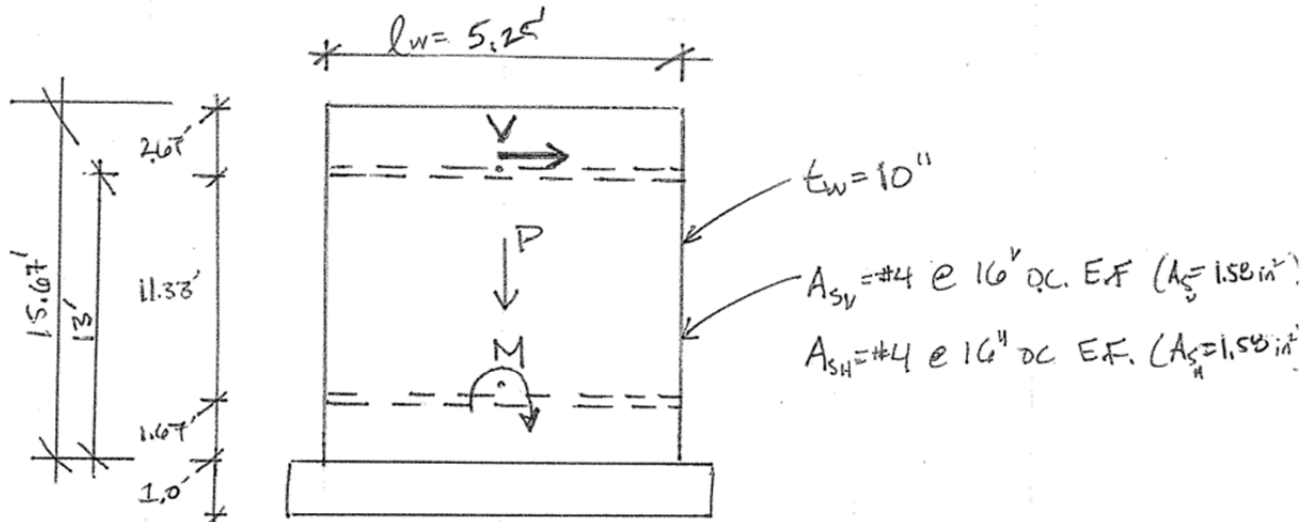
job #

11/14

date

page

SW ELEVATION



$$P = (115 \text{ PSF}) (16' / 2) (5.25') + (10' / 2 \times 150 \text{ PSF}) (15.67') (5.25') = 15.1^k$$

$$V = 179^k \text{ (QCE)}$$

$$M = (179^k) (13') - (15.1^k) (5.25' / 2) = 2287^k \text{ ft (QCE)}$$

DESIGN CRITERIA FOR SHEAR WALLS

$$K = 0.9$$

$$f'_c = 2500 \text{ psi}$$

$$f'_{ce} = 2500 \times 1.5 = 3750 \text{ psi} \quad \leftarrow \text{TABLE 10-1}$$

$$f_y = 40,000 \text{ psi}$$

$$f_{ye} = 40,000 \times 1.25 = 50,000 \text{ psi} \quad \leftarrow \text{TABLE 10-1}$$

TABLE 10-21

$$\frac{(A_s - A'_s) f_y + P}{t_w l_w f'_c} = \frac{(\overset{F(\text{ASSUMED})}{.5 \times 1.59})(40) + (15.1^k)}{(10)(5.25 \times 12)(2.5)} = .03 \leq 0.1$$

$$\frac{V}{t_w l_w \sqrt{f'_c}} = \frac{179^k \times 1000}{(10)(5.25 \times 12)(\sqrt{2500})} = 5.7 > 4 < 6$$

$$\frac{2.5 - 2.0}{6 - 4} = \frac{m - 2.0}{5.7 - 4}$$

$$m = 2.43$$

[CONTROLLED BY FLEXURE]

CHECK SHEAR IN WALL

$$Q_{CE} = 2 \sqrt{f'_c} b d + \frac{(A_{sv})(f_{ye})(d)}{(s)}$$

$$d = .8 l_w \text{ (ACI 11.9.4)}$$

$$d = .8 (525' \times 12) = 50.4''$$

$$Q_{CE} = 2(\sqrt{3750})(10'')(50.4'') + \frac{(.40)(50 \text{ ksi})(50.4'')}{(16'')}$$

$$Q_{CE} = 61.7^k + 63^k = 124.7^k$$

$$M K Q_{CE} = (2.43)(.9)(124.7^k) = 273^k > 179^k \quad \underline{\underline{OK}}$$

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CHECK FLEXURE IN WALL

- ASSUME TENSION CONTROLLED (NOT COMPRESSION CONTROLLED)

$$\omega = \frac{A_s f_y}{f'_c l_w t} = \frac{(1.58)(50)}{(3.75)(5.25 \times 12)(10)} = .033$$

$$\alpha = \frac{P_u}{f'_c l_w t} = \frac{(15.1)}{(3.75)(5.25 \times 12)(10)} = .006$$

$$\frac{c}{l_w} = \frac{\omega + \alpha}{2\omega + \alpha_1 \beta_1} = \frac{.033 + .006}{2(.033) + (.85 \times .85)} = .049$$

$$M_n = .5 A_s f_y l_w \left(1 + \frac{P}{A_s f_y}\right) \left(1 - \frac{c}{l_w}\right)$$

$$M_n = (.5)(1.58)(50)(5.25 \times 12) \left(1 + \frac{15.1}{(1.58 \times 50)}\right) (1 - .049)$$

$$M_n = 2019 \text{ k-ft}$$

$$m K Q_{CE} = (2.43)(.9)(2019) = 6,165 \text{ k-ft} > 2207 \text{ k-ft}$$

OK

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MISC CALLS:SHEAR WALLS:

$$V_j^{AVG} = \frac{1}{M_s} \left(\frac{V_j}{A_w} \right) \rightarrow \text{CHECK WORST CASE DIR AT BASE}$$

$$A_w = \left(10 \frac{1}{12} \right) (48' + 106' + 50') (1.5) = 88.35F$$

↑ CORNERS

$$v = \left(\frac{1}{4.0} \right) \left(\frac{2472^k \times 1000}{88.3 \times 144 / ft^2} \right) = 49 \text{ psi} < 100 \text{ psi} \quad \underline{\underline{OK}}$$

WALL REINF:

#4 @ 16" O.C. EW EF (TYP)

$$A_s = .2 \times 12" / 16" \text{ O.C.} \times 2 \text{ FACES} = .30 \text{ in}^2 / \text{ft}$$

$$\rho = \frac{.30 \text{ in}^2}{10" \times 12} = .0025 > .0020 \text{ (H)}$$

$$> .0012 \text{ (V)}$$

OKCOUPLING BEAMS:

$$d_{TYP} = 5'-3" = 66"$$

$$d/2 = 66" / 2 = 33" > 16" \text{ O.C. SPCG}$$

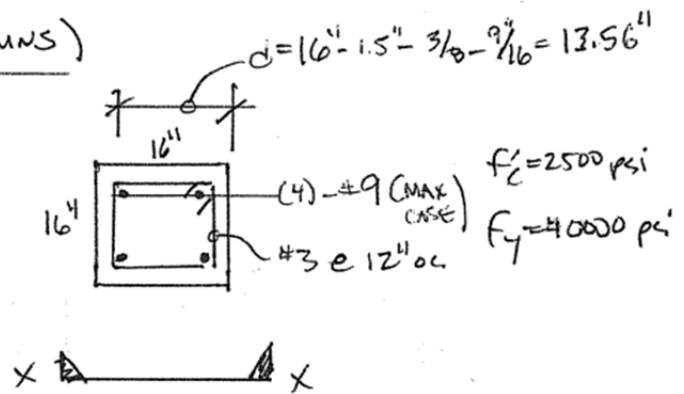
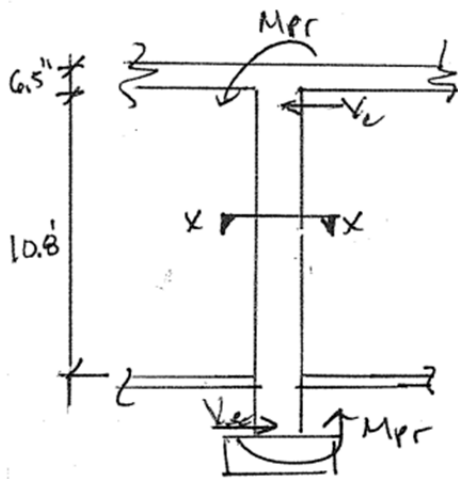
☐ NO 135° HOOKS ON STIRRUPS

ADJACENT BAYS:

$$11.33' \times .04 = 5.4" < 8"$$

N.BOK

3325 Chanate Road, Santa Rosa, CA 95404

DEFLECTION COMPATABILITY (COLUMNS)

$$V_e = \frac{M_{pr} - (-M_{pr})}{L} \leq V_n$$

$$M_{pr} = 1.25 A_s f_y (d - a/2)$$

$$a = \frac{(2 \times 1.0)(40,000)}{(0.85)(2500)(16)} = 2.35$$

$$M_{pr} = 1.25 (2 \times 1.0)(40)(13.63 - 2.35/2) = 1246 \text{ K-in}$$

$$\frac{M_{pr} - (-M_{pr})}{L} = \frac{1246 \text{ K-in} \times 2}{10.8' \times 12} = 19 \text{ K}$$

$$V_n = V_s + V_c \rightarrow 0 \text{ (NEGLECT CONCR DURING EQ EVENT)}$$

$$V_s = \frac{A_v f_y d}{s} = \frac{(0.11 \text{ in}^2 \times 2)(40 \text{ ksi})(13.56'')}{4''} = 29.9 \text{ K} > V_e$$

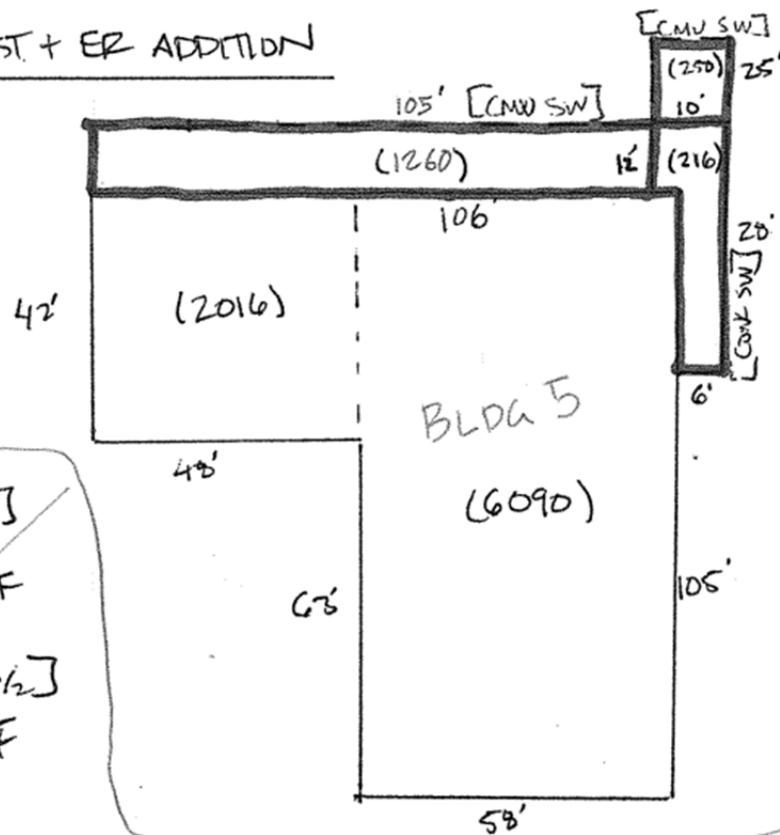
4" \nwarrow SPCC IN TOP 12" OF COLUMN

OK

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Building 6 (1956 Hospital Wing)1956 - WEST + ER ADDITION

— = BLDG 6

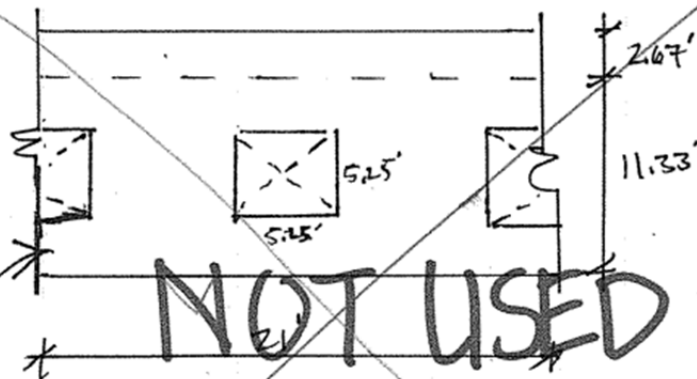


$$A_{FUE} = [2016 + 6090]$$

$$= 8754 \text{ SF}$$

$$A_{MISC} = [1050/2 + 216/2]$$

$$= 633 \text{ SF}$$

NOT USED

$$\text{Full HT WALLS} = 11.33'/2 + 2.67' = 8.33'$$

$$\text{WALLS w/ OPENINGS} = 3' + 2.67' = 5.67'$$

LATERAL BSE-1 FORCE AT ER ADDITION (CMU WALL)

$$W_R = (115 \text{ PSF}) [1260 + 216 + 250] = 199 \text{ K}$$

$$W_{\text{WALL}} = (135 \text{ PSF}) (7.63' / 12) (12' / 2) \left(\frac{55.5''}{12} \cdot 10 \right) = 24 \text{ K}$$

$$W_{\text{TOT}} = 223 \text{ K}$$

$$V = 1.395 W$$

$$V = 1.395 (223 \text{ K}) = 310 \text{ K}$$

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TIER 1 CHECKS:

ADJACENT BUILDINGS:

SEISMIC GAP @ ROOF = 4" PER STRUCTURAL DRAWINGS

$$\text{GAP REQD} = 0.04(12' \cdot 12'/1') = 5.75"$$

 \therefore NONCOMPLIANT PER TIER 1 \leftarrow

SHEAR STRESS CHECK:

$$V = 311^k$$

$$V_{ALL} = 70 \text{ PSI}$$

$$\tau = (1/M_s)(V/A_w) = (1/4)(311^k / (10 \text{ WALLS} \cdot 55" \cdot 6")) = 24 \text{ PSI} \ll 70 \text{ PSI}$$

ONLY PARTIALLY 6"
 \therefore CONSERVATIVE

 \therefore COMPLIANT PER TIER 1 \leftarrow

REINFORCING STEEL CHECK:

VERTICAL BARS:

$$(8) \#5 + (1) \#4 = (8)(.31) + (.20) = 2.68" / (55 \cdot 7\frac{3}{8}) = 0.0063$$

PARTIALLY 6"
 \therefore CONSERVATIVE

0.0007 ✓

HORIZONTAL BARS:

$$\#3 @ 16" OC = 0.11 / (16 \cdot 7\frac{3}{8}) = 0.0009 > 0.0007 \checkmark$$

$$\text{TOTAL: } 0.0063 + 0.0009 = 0.0072 > 0.002 \checkmark$$

 \therefore COMPLIANT PER TIER 1 \leftarrow

TIER 1 CHECKS (CONTINUED):

WALL ANCHORAGE CHECKS:

$$T_c = \psi S_x S_w p A_p$$

(4-13)

$$\hookrightarrow \psi = 1.2 \text{ (LIFE SAFETY)}$$

$$\hookrightarrow S_x S_w = 0.997 = 1$$

$$\hookrightarrow W_p = 135 \text{ PCF } (7.625/\text{ft}) = 85 \text{ PSF}$$

$$\hookrightarrow A_p = 12''/12'' (12'/2) = 6 \text{ SF}$$

$$\therefore T_c = 1.2(1)(85)(6.0) = 612^\#$$

3 @ 12" OC DOWELS IN SHEAR

SHEAR CAPACITY OF #3 BAR IN SHEAR FRICTION

$$V_c = 0.6(A_g)(f_y)$$

$$= 0.6(.11)(33)$$

$$= 2.2^k \gg T_c = 612^\#$$

e. COMPLIANT PER TIER 1 ←

3325 Chanate Road, Santa Rosa, CA 95404

Building 7 (1936 Original Hospital Building)**DESIGN CRITERIA****Assumed/Default Materials**Concrete: $f'_c = 2,000$ psi @28 days

Reinf. Steel: Grade 33

Structural Steel: WF Shapes ASTM A-9 (33ksi)

C and L Shapes, and Plates ASTM A-9 (33ksi)

LGMF: 33ksi



Stamp

DESIGN LOADING

	SLOPED ROOF		2nd FLOOR	1st FLOOR	INTERIOR WALLS	EXTERIOR WALLS
LIVE LOADS (PSF)						
DEAD LOADS (PSF)						
Roofing Spanish Clay Tile	19.0					
Fin. Floor		Linoleum	1.0	1.0		
Sheathing 2.5" Conc	30.2	2.5" Conc	30.2	30.2		
Joists/Truss OWJ @ 32"oc	2.6	OWJ @ 32"oc	2.6	2.6		
Beams						
Ceiling Plaster/Mtl Lath	10.0	Plaster/Mtl Lath	10.0			
Insulation	1.0					
HVAC	1.0		1.0	1.0		
Partitions						
Sprinklers	1.5		1.5			
Misc. Incl 5:12 Rf Slope (x0.08 = 4.2)	6.7		3.7	3.2	Plaster ES	Plaster ES
DEAD LOADS (PSF)	72.0		50.0	38.0	20.0	20.0
TOTAL LOADS (PSF)	72.0		50.0	38.0		

Foundation:

Design Dead + Live Bearing = **3000** psf
 Design Dead + Wind/Seismic Bearing = **4000** psf

Geotechnical Report by:

Rutherford & Chekene
 2002-112G
 12/20/2002

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Tier 1 Lateral - Diaphragm Weight

11/11/14

Diaphragm Weight Information:

Level	Area (sq ft)	Diaphragm Unit Weight (psf)	Diaphragm Weight (lb)	Wall Unit Weight (psf)	Wall Trib Width (ft)	Wall Length (ft)	Wall Weight (lb)	Level Weight (lb)
ROOF	18565	72	1,337	20	5.67	2035	231	1,567
2nd	18565	50	928	20	11.33	2035	461	1,389
1st	18565	38	705	20	7.17	2035	292	997
Σ			2,970				984	3,954

Tier 1 Lateral - Story Force Distribution

11/11/14

Seismic Story Force Distribution based on ASCE 41 Tier 1

$S_{XS} = 0.997$

$T_a \text{ Period} = 0.26$

$k = 1$

(4.5.2.2)

$I_{seismic} = 1.00$

$\rho = 1.0$

ULT

$V(ULT) = 1.196$

Base V (ULT) = 4,731

Pseudo Seismic Force (4.5.2.1)

Story Force Vertical Distribution (ASCE 41-13 4.5.2.2)

Level	w_x	h_x (ft.)	h_x^k	$w_x h_x^k$	F_x , ULT	$Cv_x\%$
ROOF	1,567	30.5	30.5	47807	3,145	66.5
2nd	1,389	14.8	14.8	20605	1,356	28.7
1st	997	3.5	3.5	3491	230	4.9
Σ	3,954			71902	4,731	

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Lateral - Story Force Distribution

11/11/14

Seismic Story Force Distribution based on ASCE 7

$$\begin{aligned}
 S_{DS} &= 1.651 & T_a \text{ Period} &= 0.26 & k &= 1 & (12.8.3) \\
 I_{seismic} &= 1.00 & \rho &= 1.0 & R &= 3.25 \\
 V(ULT) &= 0.508 & \text{Base V (ULT)} &= 2,009 & \text{Base Shear (12.8-1)} & &
 \end{aligned}$$

Story Force Vertical Distribution (ASCE 7-10 12.8.3)

Level	w_x	h_x (ft.)	h_x^k	$w_x h_x^k$	F_x , ULT	$Cv_x\%$
ROOF	1,567	30.5	30.5	47807	1,336	66.5
2nd	1,389	14.8	14.8	20605	576	28.7
1st	997	3.5	3.5	3491	98	4.9
Σ	3,954			71902	2,009	

Vertical Diaphragm Distribution (ASCE 7-10 12.10.1.1)

Level	w_x	Σw_x	F_x	ΣF_x	F_{px} , ULT
ROOF	1,567	1,567	1,336	1,336	1,035
2nd	1,389	2,957	576	1,911	898
1st	997	3,954	98	2,009	507
Σ	3,954		2,009		

Where $F_{pmin} = \rho * 0.2 SDS I W_x, ULT$
 $F_{pmax} = \rho * 0.4 SDS I W_x, ULT$

Tier 1 Diagonal Brace Check

11/11/14

Braced Wall Panel Diagonal Rods

$M_s = 3.0$

$0.50F_y = 16.50$

Story Shears

	1st Story	2nd Story
$V_j =$	4,501	3,145

Longitudinal Direction

Number N_{br}	Bay s (ft)	Diag L_{br} (ft)	1st Story A_{br} (in ²)	2nd Story A_{br} (in ²)
15	14.0	18.01	0.6	0.31
5	14.0	18.01	0.2	0.2
7	8.0	13.87	0.31	0.31
10	5.0	12.38	0.6	0.31
Total = 37	10.43	15.71	0.49	0.30

(Weighted Averages)

$L_{br} / M_s * s * N_{br} = 0.0136 \quad 0.0136$

$V_j * L_{br} / A_{br} * M_s * s * N_{br} = 124.3 \quad 144.6$

$D/C \text{ at Tier 1 Level} = 7.53 \quad 8.76$

Transverse Direction

Number N_{br}	Bay s (ft)	Diag L_{br} (ft)	1st Story A_{br} (in ²)	2nd Story A_{br} (in ²)
22	14.0	18.01	0.6	0.31
4	8.0	13.87	0.31	0.31
Total = 26	13.08	17.37	0.56	0.31

(Weighted Averages)

$L_{br} / M_s * s * N_{br} = 0.0170 \quad 0.0170$

$V_j * L_{br} / A_{br} * M_s * s * N_{br} = 138.0 \quad 172.8$

$D/C \text{ at Tier 1 Level} = 8.37 \quad 10.47$

Tier 1 Column Overturning Check

11/11/14

Braced Wall Panel Edge Studs

$M_s = 3.0$

$0.30F_y = 9.90$

Base Shear

$V_j = 4,501$

Longitudinal Direction

Number	Bay	Tot Height	1st Story
n_f	L	h_n	A_{col}
	(ft)	(ft)	(in ²)
15	14.0		
5	14.0		
7	8.0		
10	5.0		
Total =	37	10.43	24.00
			0.75
		(Weighted Averages)	
		$2/3 * h_n / M_s * L * n_f =$	0.0138
		$2/3 * V * h_n / M_s * L * n_f * A_{col} =$	82.9
		D/C at Tier 1 Level =	8.38

Transverse Direction

Number	Bay	Tot Height	1st Story
n_f	L	h_n	A_{col}
	(ft)	(ft)	(in ²)
22	14.0		
4	8.0		
Total =	26	13.08	24.00
			0.75
		(Weighted Averages)	
		$2/3 * h_n / M_s * L * n_f =$	0.0157
		$2/3 * V * h_n / M_s * L * n_f * A_{col} =$	94.1
		D/C at Tier 1 Level =	9.51

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Tier 1 Brace Check

11/11/14

New Braced Frames

$$M_s = 6.0$$

$$0.50F_y = 21.00$$

Story Shears

	1st Story	2nd Story
$V_j =$	4,501	3,145

Longitudinal Direction

Number	Bay	Diag	1st Story	2nd Story
N_{br}	s	L_{br}	A_{br}	A_{br}
	(ft)	(ft)	(in ²)	(in ²)
36	12.0	16.50	2.09	2.09

Total =	36	12.00	16.50	2.09	2.09
	(Weighted Averages)				
	$L_{br} / M_s * s * N_{br} =$	0.0064	0.0064		
	$V_j * L_{br} / A_{br} * M_s * s * N_{br} =$	13.7	9.6		
	D/C at Tier 1 Level =	0.65	0.46		

Transverse Direction

Number	Bay	Diag	1st Story	2nd Story
N_{br}	s	L_{br}	A_{br}	A_{br}
	(ft)	(ft)	(in ²)	(in ²)
36	12.0	16.50	2.09	2.09

Total =	36	12.00	16.50	2.09	2.09
	(Weighted Averages)				
	$L_{br} / M_s * s * N_{br} =$	0.0064	0.0064		
	$V_j * L_{br} / A_{br} * M_s * s * N_{br} =$	13.7	9.6		
	D/C at Tier 1 Level =	0.65	0.46		

Tier 1 Column Overturning Check

11/11/14

New Braced Frames Columns

$$M_s = 6.0$$

$$0.30F_y = 13.80$$

Base Shear

$$V_j = 4,501$$

Longitudinal Direction

Number	Bay	Tot Height	1st Story
n_f	L	h_n	A_{col}
	(ft)	(ft)	(in ²)
18	12.0		

Total =	18	12.00	24.00	6.18
---------	----	-------	-------	------

(Weighted Averages)

$$2/3 * h_n / M_s * L * n_f = 0.0123$$

$$2/3 * V * h_n / M_s * L * n_f * A_{col} = 9.0$$

$$D/C \text{ at Tier 1 Level} = 0.65$$

Transverse Direction

Number	Bay	Tot Height	1st Story
n_f	L	h_n	A_{col}
	(ft)	(ft)	(in ²)
18	12.0		

Total =	18	12.00	24.00	6.18
---------	----	-------	-------	------

(Weighted Averages)

$$2/3 * h_n / M_s * L * n_f = 0.0123$$

$$2/3 * V * h_n / M_s * L * n_f * A_{col} = 9.0$$

$$D/C \text{ at Tier 1 Level} = 0.65$$

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Building 8 (1956 Kitchen/Storage Building)

SEISMIC

$$V = C S_a W$$

 $C = 1.0$ flexible diaph

$$S_a = S_{x1/T} \leq S_{x5}$$

$$S_s = 0.996 \quad F_a = 1.002$$

$$S_1 = 0.389 \quad F_v = 1.411$$

$$S_{x5} = 0.997$$

$$S_{x1} = 0.549$$

$$T = C_t h_n^{\beta} = 0.02(12)^{0.75} = 0.13$$

$$S_a = 0.549/0.13 = 4.2 \leq 0.997$$

$$\Rightarrow S_a = 0.997 \Rightarrow 1.0$$

Bldg. Wgt.

Roof Wgt per orig drawing
= 26 psf max

8" EXT CONC. WALLS 100psf
w/ 12"x17" CONC BM

neglect opns & bms/coi
conservative wgt.

12" SQ COL @ 18' o.c.

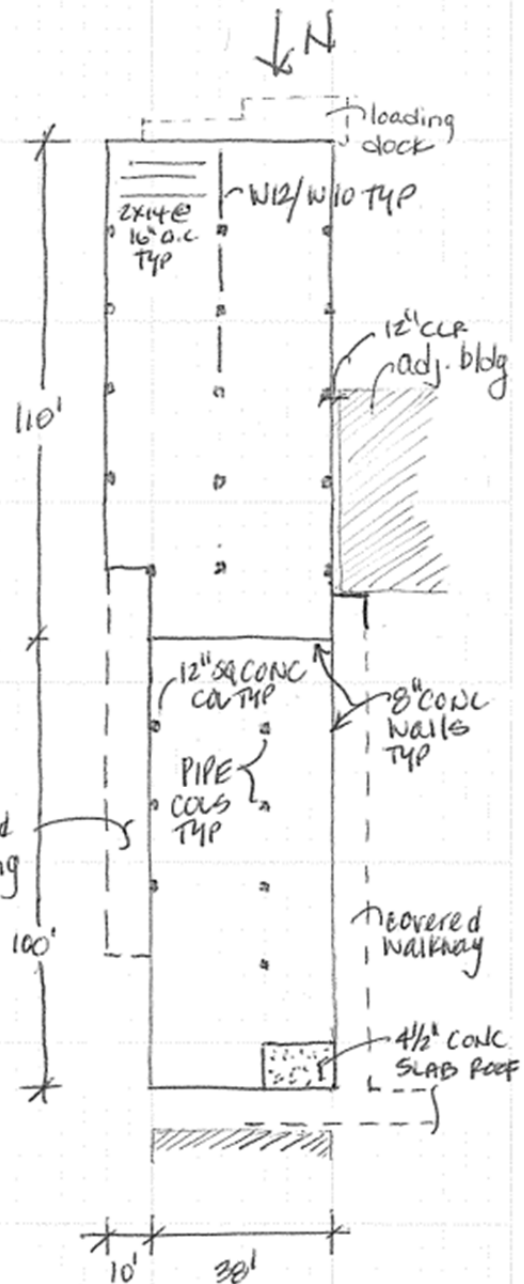
$$W = 26 \text{ psf} (210' \cdot 48')$$

$$+ 100 \text{ psf} (12.3' (420' + 96' + 38' + 15.7' + 10' + 9' + 11.7'))$$

$$+ 150 \text{ pcf} (4\frac{1}{2}'/12' (15.7' (10)))$$

$$= 262^k + 739^k + 9^k = 1010^k$$

$$V = 1.0(1.0 \times 1010^k) = 1010^k$$



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CONC SHEARWALLS:

REINF RATIO

8" WALL w/ #4 @ 10" O.C. EW

$$\rho_h = \rho_v = \frac{0.2}{8(10)} = 0.0025 > 0.002 \therefore \text{OK}$$

WALL STRESS

$$v_j = \frac{1}{m} \left(\frac{V}{A_w} \right)$$

$$m = 4.0$$

$$A_w = 8"(12)(48' + 65') = 10,850 \text{ in}^2 \quad \text{NS}$$

$$8"(12)(81') = 7780 \text{ in}^2 \quad \text{EW governs}$$

$$v_j = 32 \text{ psi} < 100 \text{ psi} \therefore \text{OK}$$

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WALL ANCHORAGE

CAP OF TYP WALL ANCHORAGE
TO PERPENDICULAR FRAMING(4) 16d NAIL CAP w/ $\phi = 1.0$

$$CAP OF 16d = 3.32(0.65)(180\text{lbs}) = 388\text{lbs}$$

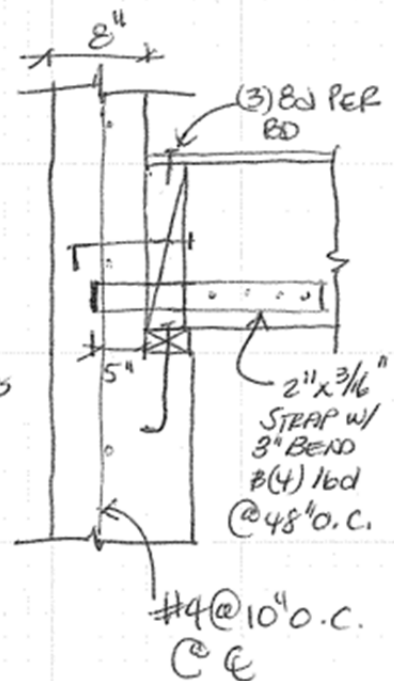
$$\Rightarrow 4(388) = 1.55^k = Q_{CE}$$

GOVERNS

CAP OF 2" x 3/16" R

$$= A_s F_y \quad F_y = 33\text{ksi} = Q_{CL}$$

$$= 2(3/16)(33\text{ksi}) = 12.4^k$$



CAP OF STRAP IN CONC

 $f'_c = 2500\text{ psi}$ per orig drawings $\phi = 1.0$ §10.3.6.1

Lower bound cap is the lesser of

 $0.75\phi N_{cb}$, $0.75\phi N_{pn}$ side face blowout NA

$$N_{cb} = \frac{A_{nc}}{A_{nc0}} \gamma_{ed} \gamma_c \gamma_{cp} N_b$$

$$A_{nc0} = 9h_{ef}^2 = 225 = A_{nc}$$

$$h_{ef} = 5"$$

$$\gamma_{ed} = 1.0 \quad \gamma_c = 1.0 \quad \gamma_{cp} = 1.0 \quad k_c = 24 \quad \lambda_a = 1.0$$

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} = 13.4^k$$

$$N_{pn} = \gamma_c N_p$$

$$\gamma_c = 1.0$$

$$N_p = 0.9 f'_c e_h d_a = 6.8^k$$

$$d_a = 1 \quad \text{limited by } 1/3 e_h \quad e_h = 3$$

$$\Rightarrow Q_{CL} = 0.75(6.8^k) = 5.1^k$$

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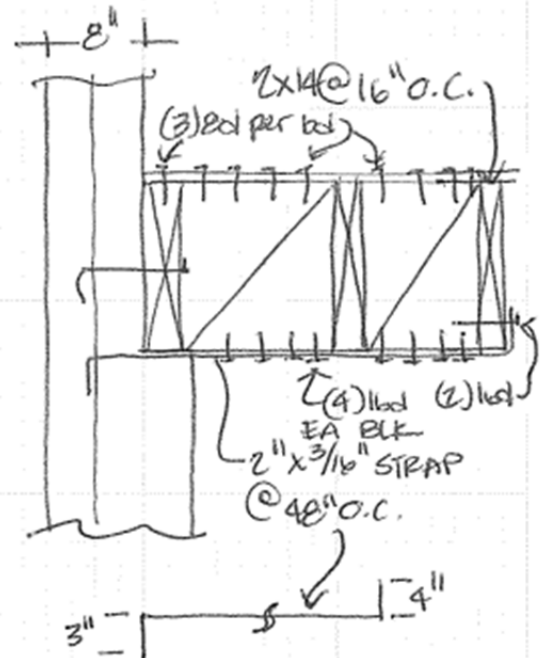
WALL ANCHORAGE

CAP OF TYP WALL ANCHORAGE
TO PARALLEL FRAMING

$$(8) 16d \text{ NAIL CAP} \\ = 8(388) = \underline{3.1^k} \\ \text{GOVERNS}$$

$$\text{CAP OF } 2" \times 3/16" \text{ PL} \\ = 12.4^k$$

see prev pg



CAP OF STRAP IN CONG

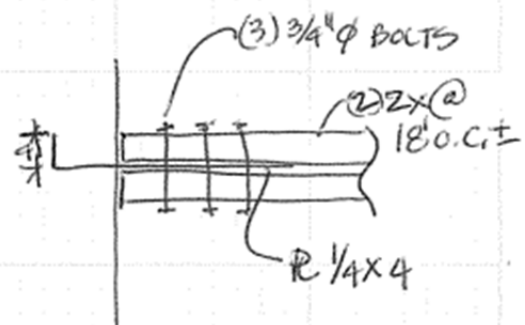
$$= 5.1^k$$

see prev. pg

CAP OF GIRDER ANCHORAGE
TO PERP FRAMING @ 18' O.C.

$$3/4" \phi \text{ BOLTS IN WOOD} \\ = 3.32(0.65)(3150 \text{ lbs}) = 6.8^k \\ \Rightarrow (3) 6.8^k = 20.4^k$$

$$0.75 N_{pn} = 0.75(1.0)(0.9)(2500 \text{ psi})(1/3 \cdot 4")(4") \\ = 9^k \\ \underline{\text{GOVERNS}}$$



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Out of plane wall anchorage forces

$$T_c = \psi S_{xs} W_p A_p$$

$$A_p = \left(\frac{1}{2}(11') + 1'4'' \right) = 6.2 \text{ ft} / \text{ft}$$

$$\text{w/ } 4' \text{ o.c. anchors } A_p = 24.8 \text{ ft}^2$$

$$W_p = 100 \text{ psf}$$

$$\psi = 1.2 \quad \text{LS Flex diaph.}$$

$$S_{xs} = 1.0$$

$$T_c = 3.0 \text{ k}$$

$$13.4 \text{ k}$$

w/ anchors @ 4' o.c.

w/ anchors @ 16' o.c.

$$\text{anchors } \parallel \text{ framing } Q_{CE} = 3.1 \text{ k} > T_c = 3.0 \text{ k} \quad \therefore \text{OK}$$

$$\text{anchors } \perp \text{ framing } Q_{CE} = 1.6 \text{ k} < T_c = 3.0 \text{ k} \quad \therefore \text{NG}$$

$$\text{@ girder } 9 \text{ k} < 13.4 \text{ k} \quad \therefore \text{NG}$$

Tier 2 check

$$F_p = 0.4 S_{xs} k_a k_h \chi W_p \geq 0.2 k_a \chi W_p$$

$$k_h = 1.0 \quad \chi = 1.3$$

$$k_a = 1.0 + \frac{L_f}{100} \leq 2.0 \quad \text{w/ } L_f = 110, 100 \perp \text{ frng}$$

$$= 2.0 \perp \text{ frng}$$

$$= 1.4 \parallel \text{ frng}$$

$$F_p = 1.04 (S_{xs}) W_p = 2.6 \text{ k} > Q_{CE} = 1.6 \text{ k} \quad \therefore \text{NG}$$

$$= 0.73 (S_{xs}) W_p = 1.8 \text{ k} < Q_{CE} = 3.1 \text{ k} \quad \therefore \text{OK}$$

INCREASE NAILING ^{TO (8 lbd)} IN (E) STRAPS \perp FRMG TO INCREASE CAP

TIER 2 CHECK DIAPHRAGM

$$V = C_1 C_2 C_m S_a W$$

$$C_1 C_2 = 1.1 \quad \text{per table 7-3}$$

$$C_m = 1.0 \quad 7-4$$

$$S_a = 1.0$$

$$V = 1.1W$$

$$Q_{CE} = 600 \text{ p/f}$$

$$m = 2.0$$

$$w/ \ 110/38 = 2.9 \text{ aspect ratio}$$

$$V = \left[\frac{110'}{2} (26 \text{ psf} (48') + 100 \text{ psf} (6.2' \cdot 2)) \right] / 38'$$

$$= 3600 \text{ p/f} = Q_{ud}$$

$$k = 1.0 \quad - \text{for diaphragm visible during site investigation}$$

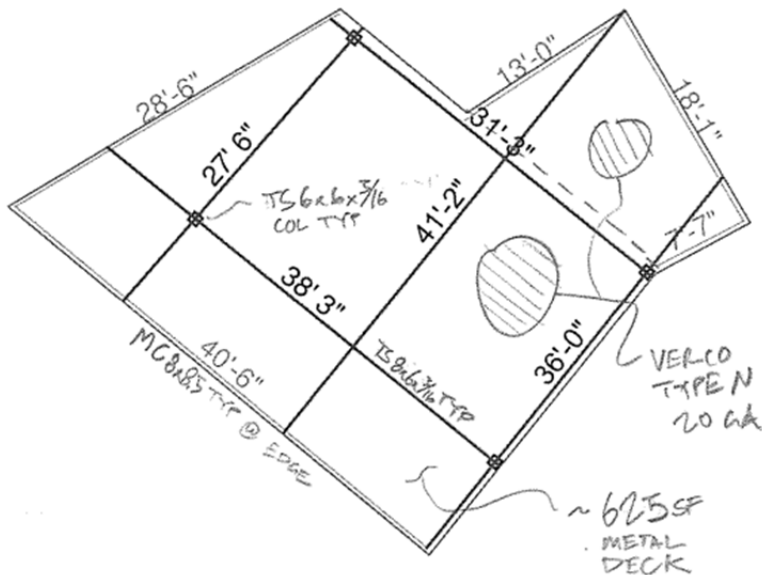
$$m k Q_{CE} = 2.0 (1.0 \times 600 \text{ p/f}) \quad \therefore \quad Q_{ud}$$

$$m k Q_{CE} = 1200 \text{ p/f} < Q_{ud} = 3600 \text{ p/f} \therefore \text{NG}$$

diaphragm is NOT ADEQUATE
PER TIER 2 ANALYSIS

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Building 9 (1987 Ambulance Canopy)

TIER 1 CHECKS:

$$f_y = 37 \text{ ksi}$$

$$f_{yc} = 37(1.05) \text{ (TABLE 93)} \\ = 38.9 \text{ ksi}$$

SEISMIC WEIGHT, W:

$$\text{ROOF: } 625 \text{ SF } (3 \text{ PSF} + 1 \text{ PSF, MISC})$$

$$\text{BMS: } (27.5 + 38.25 + 41 + 36 + 31.25)(17.1 \text{ PLF})$$

$$\text{EDGE: } (28.5 + 40.5 + 13 + 18 + 7.5)(8.5 \text{ PLF})$$

$$\text{COL: } 11\frac{1}{2} (4 \text{ COLS}) (14.5 \text{ PLF})$$

$$= 2,500 \#$$

$$= 2,975 \#$$

$$= 914 \#$$

$$= 319 \#$$

$$\underline{6,708 \# = W}$$

$$(6708 - 319) / 625 = 10 \text{ PSF} = w$$

PSEUDO SEISMIC FORCE:

$$T = C_t h_n^{\frac{1}{4}} = 0.035 (11')^{0.25} = \underline{0.238 \text{ s}}$$

$$C_{TIER 1} = \underline{1.0} \text{ (TABLE 4-8 FOR FLEX DIA)}$$

$$S_a = S_{x1} / T = 0.548 / 0.238 = 2.30 \Rightarrow S_{x5} = 0.997$$

$$\therefore S_a = \underline{0.997}$$

$$V = C S_a W = 1.0 (0.997) (6,708) = \underline{6,690 \#}$$

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TIER 2 CHECKS:

$$V = C_1 C_2 C_m S_a W$$

$$\hookrightarrow C_1 C_2 = 1.1 \quad (\text{TABLE 7-3})$$

$$\hookrightarrow T = .238 < .3 + M_{max} < 2$$

$$\hookrightarrow C_m = 1.0 \quad (\text{TABLE 7-4})$$

$$\hookrightarrow 1-2 \text{ STORIES}$$

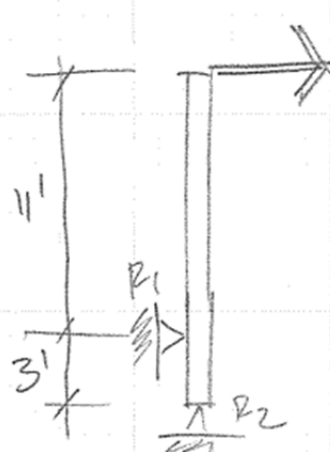
$$\hookrightarrow S_a = 0.997 \quad (\text{SEE TIER 1})$$

$$\therefore V = 1.1 (1.0) (.997) W$$

$$= 1.1 W$$

$$V_{\text{BASE TIER 2}} = 1.1 (6,708^{\#})$$

$$= \underline{\underline{7.4^k}}$$



$$V/4 \text{ COLUMNS} = 1.85^k$$

$$M_{max} = 1.85^k (11') = \underline{\underline{20.4^k \text{ FT}}}$$

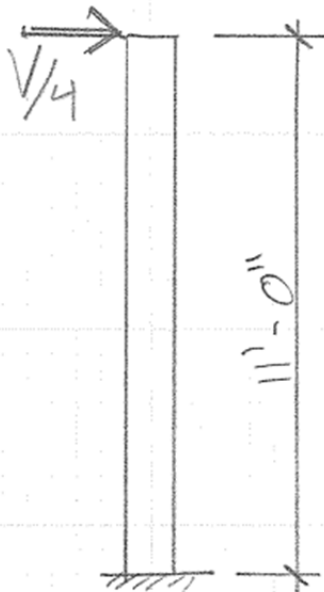
$$R_1 = 1.85 (3) / 3 = 8^k$$

$$R_2 = R_1 - 1.85^k = 6.2^k$$

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TIER 1 CHECKS:

DRIFT CHECK:



$$\Delta = PL^3/3EI$$

$$= (6,690/4)(11.12)^3 / 3(29000)(22.3)$$

$$= 1.98''$$

$$\therefore \text{DRIFT} = \Delta/L = 1.98/(11.12) = 0.015$$

$$< 0.025 \quad \checkmark$$

$$\therefore \text{COMPLIANT}$$

FLEXURAL STRESS CHECK: (PER 4.5.3.9 SM)

$$f = M/M_s z = V(u)/4M_s z \quad \text{MOD FACTOR FOR LS}$$

$$= 6,690(11.12'')/4(.8)(8.63)$$

$$= 31,977 \text{ PSI} = 32 \text{ KSI} < f_y = 37 \text{ KSI} \quad \checkmark$$

$$\therefore \text{COMPLIANT}$$

COMPACT MEMBERS CHECK:

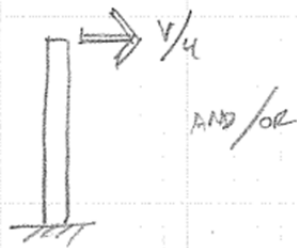
$$b/t_{\text{LIMIT MOD DUCTILE}} = 0.64\sqrt{E/F_y} = 0.64\sqrt{29000/37} = 17.9$$

$$b/t_{\text{TS 6x6x3/16}} = 31.5 > 17.9 \quad \times \quad \therefore \text{NONCOMPLIANT}$$

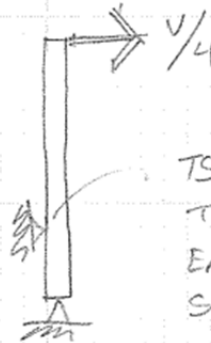
TIER 2 ADJACENT BUILDINGS:

SEISMIC GAP = 2"
(TO STRUCTURAL ELEMENTS)

CANTILEVERED COLUMN DEFLECTION MODELS



AND/OR



TS COLUMNS ARE PINNED TO THE SLAB, PARTIALLY ENCASED IN CONC PILASTER BELOW SLAB, AND PINNED TO A PAD FTG

ACTUAL DEFLECTION IS BETWEEN THE TWO MODELS BUT CHECK THE FIXED BASE DEFLECTION FIRST CONSERVATIVELY

$$\Delta_{\text{FIXED (MIN)}} = PL^3/3EI$$

$$= 1.85^k (11' \cdot 12'')^3 / 3(29000)(22.3) = 2.19'' > 2'' \times$$

ACTUAL DEFLECTION IS EXPECTED TO BE GREATER THAN Δ_{FIXED}

∴ THE SEISMIC GAP TO ADJACENT BUILDINGS IS NON-COMPLIANT PER THE TIER 2 ANALYSIS

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LOWER BOUND COLUMN STRENGTH (P_{CL}):TS 6x6x 3/16, $L = 11'$ CANTEVERED COLUMN

$$\hookrightarrow A_g = 4.27 \text{ in}^2, r = 2.36 \text{ in}$$

$$KL = 2.10 (11' \cdot 12') = 277''$$

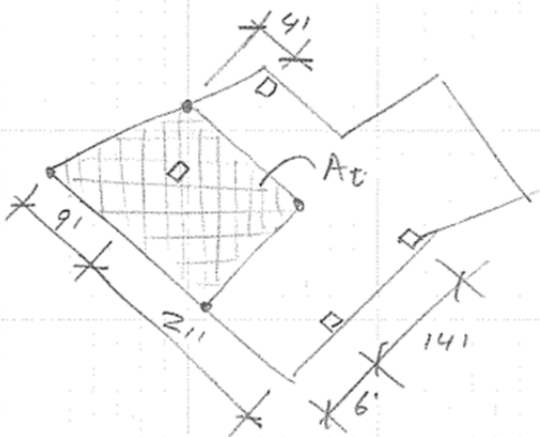
$$KL/r = 277 / 2.36 = 117$$

$$F_c = \pi^2 E / (KL/r)^2 = \pi^2 (29,000) / (117)^2 = 20.9 \text{ ksi}$$

$$4.71 \sqrt{E/F_y} = 4.71 \sqrt{29,000/37} = 132 > KL/r = 117$$

$$\therefore F_{cr} = (0.658^{F_y/E}) F_y = (0.658^{37/20.9}) 37 = 17.6 \text{ ksi}$$

$$\therefore P_{CL} = k F_{cr} A_g = (17.6 \text{ ksi}) (4.27) = \underline{56 \text{ k}}$$

MAXIMUM COLUMN AXIAL DEMAND (P):

$$DL = 10 \text{ PSF} \quad LL = 20 \text{ PSF}$$

$$A_{TRIB} = (14/2 + 6) \left(\frac{(21/2 + 9) + (21/2 + 4)}{2} \right) = 221 \text{ SF}$$

$$\therefore P = 1.1 (10 + 20) \text{ PSF} (221 \text{ SF})$$

$$P = \underline{7.3 \text{ k}}$$

$$P/P_{CL} = 7.3 / 56 = 0.13 < 0.2$$

M-FACTOR CALCULATION (TABLE 9-4):

FOOTNOTE a

$$P/P_{OL} = 0.13 < 0.2$$

$$b/t = 31.5$$

$$110/\sqrt{f_{yc}} = 110/\sqrt{37(1.05)} = 17.6 < b/t = 31.5$$

$$190/\sqrt{f_{yc}} = 190/\sqrt{37(1.05)} = 30.5 < b/t$$

$$300/\sqrt{f_{yc}} = 300/\sqrt{37(1.05)} = 48.1 > b/t$$

$$460/\sqrt{f_{yc}} = 460/\sqrt{37(1.05)} = 73.8 > b/t$$

$$\therefore m = \underline{1.25} \text{ (LIFE SAFETY)}$$

EXPECTED FLEXURAL STRENGTH (M_{CE}):

$$M_{CE} = k_z f_{yc} = (75) 9.24 (37^{kz} \cdot 1.05) = \underline{269 \text{ k-in}}$$

FLEXURAL DEMAND (M):

$$M_{max} = 20.4 \text{ kft} (12 \text{ in/ft}) = 245 \text{ k-in} \quad (\text{SEE BASE SHEAR CALC})$$

ACCEPTANCE CRITERIA:

$$P/P_{OL} = 0.13 < 0.2$$

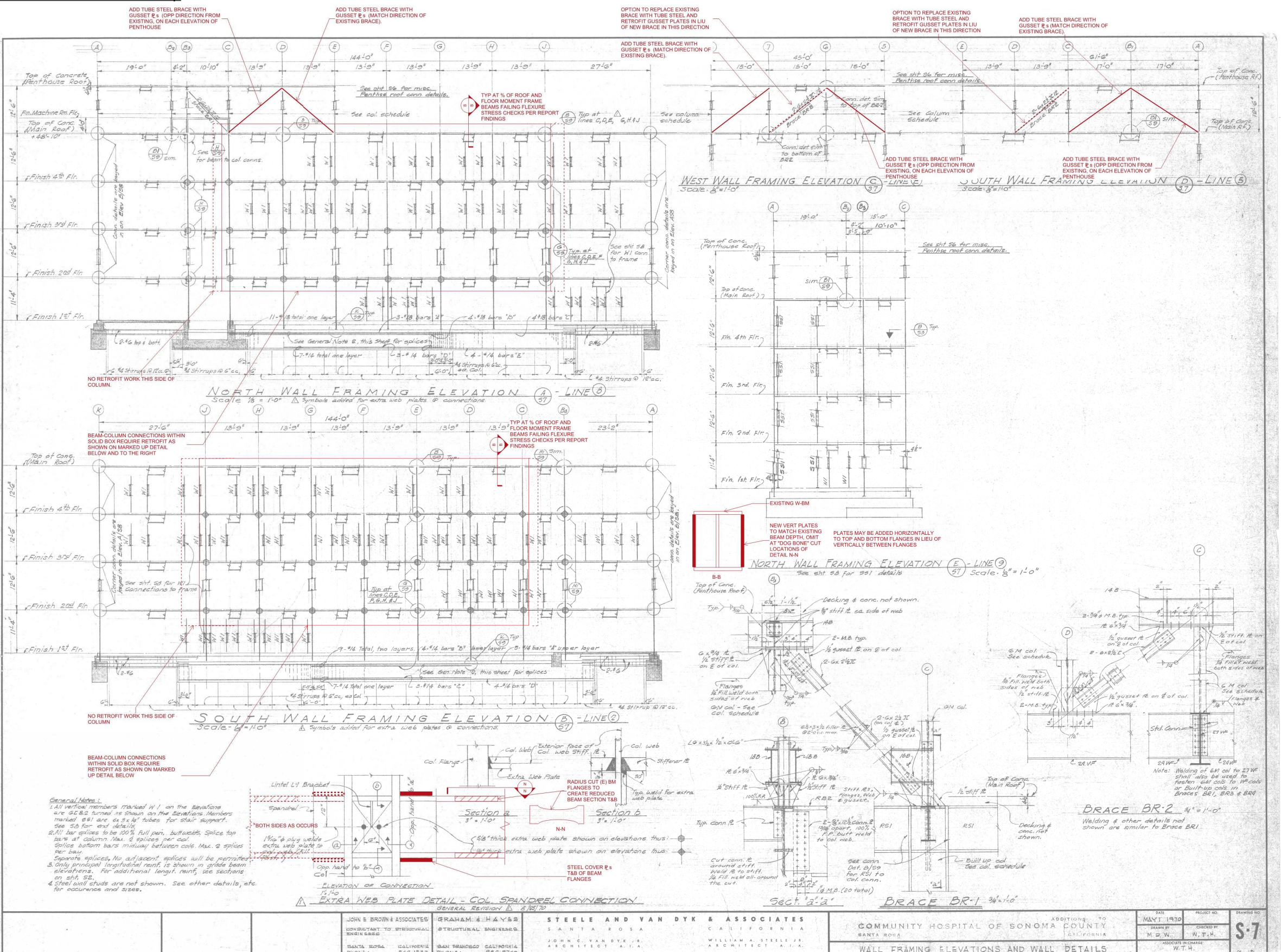
$$\therefore P/2P_{OL} + \frac{M}{m M_{CE}} \leq 1.0$$

$$0.13/2 + 245/1.25(269) = 0.79 < 1.0 \quad \checkmark$$

\therefore NON-COMPACT COLUMNS
ARE COMPLIANT
AT THE LIFE SAFETY
PERFORMANCE LEVEL

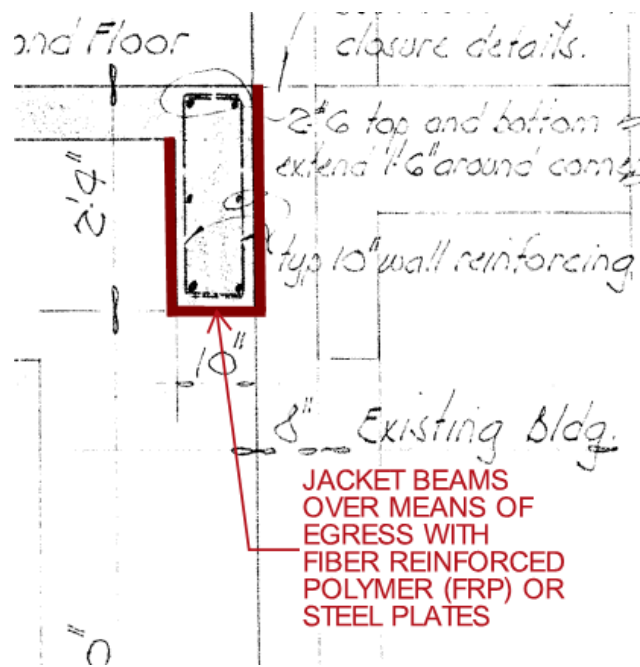
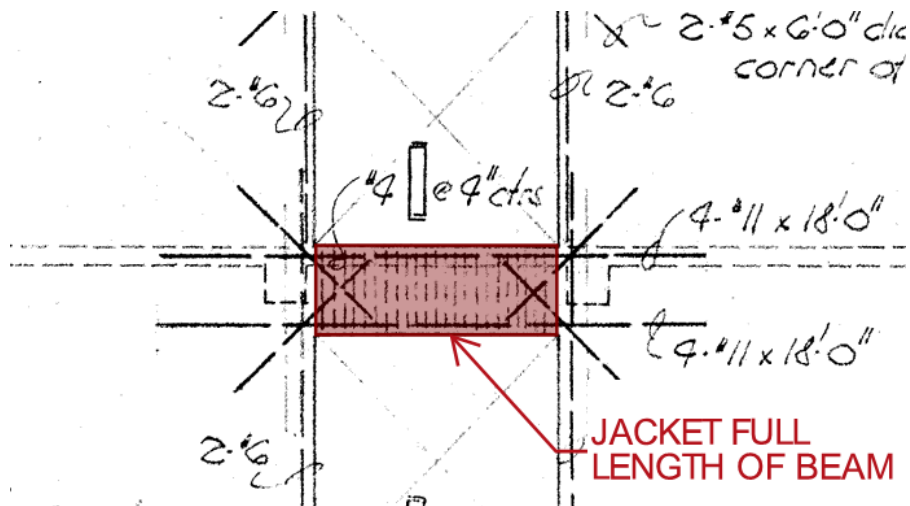
APPENDIX G – SCHEMATIC REPAIR DETAILS

Building 2 1972 4-Story: Structural Elevation Markup



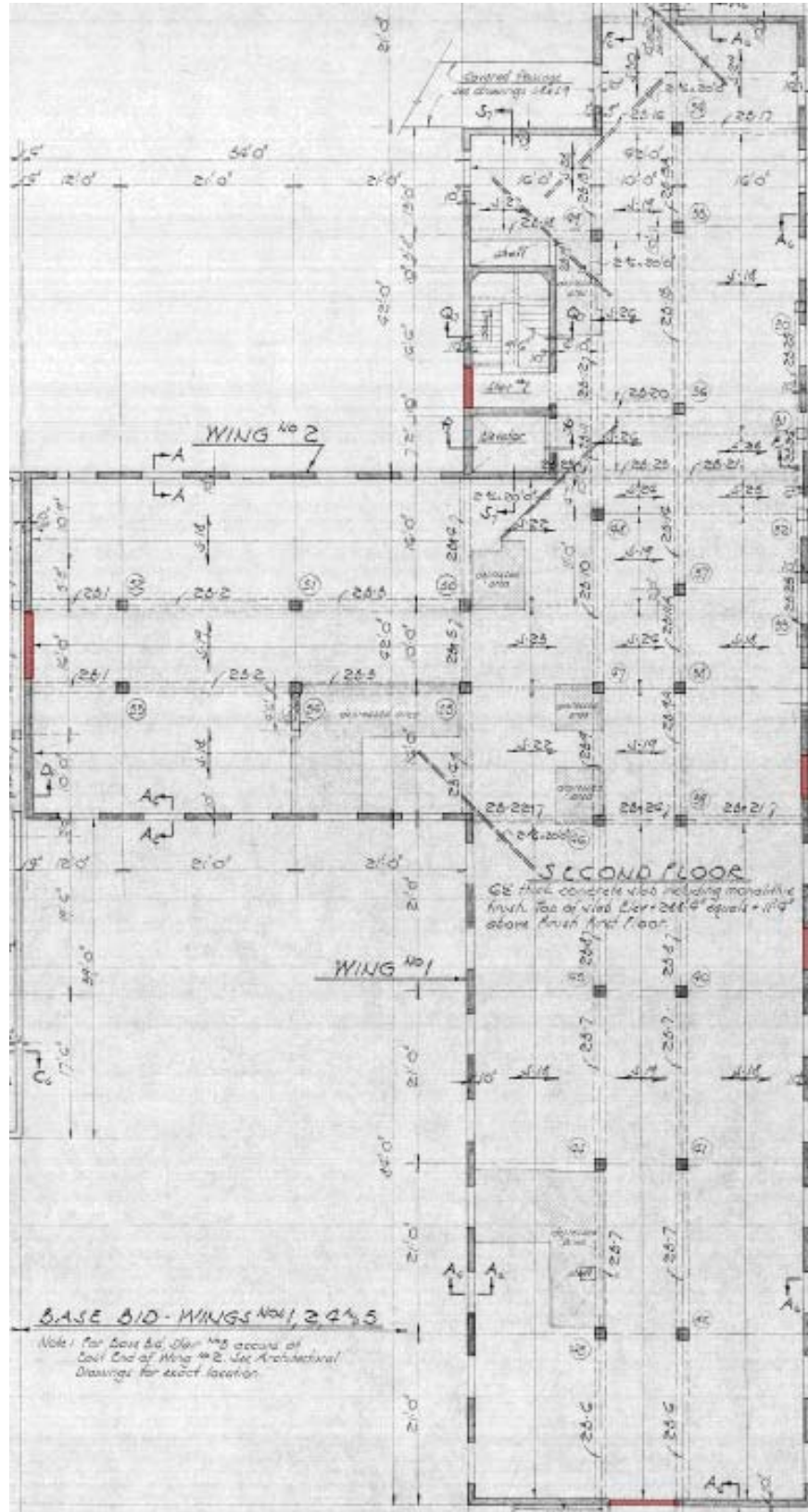
ZFA STRUCTURAL ENGINEERS

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Buildings 3, 4 and 5 (1956 Hospital Wing)TYPICAL BEAM SECTIONTYPICAL BEAM ELEVATION

- SEE FOLLOWING PLAN MARK-UPS FOR BEAM LOCATIONS

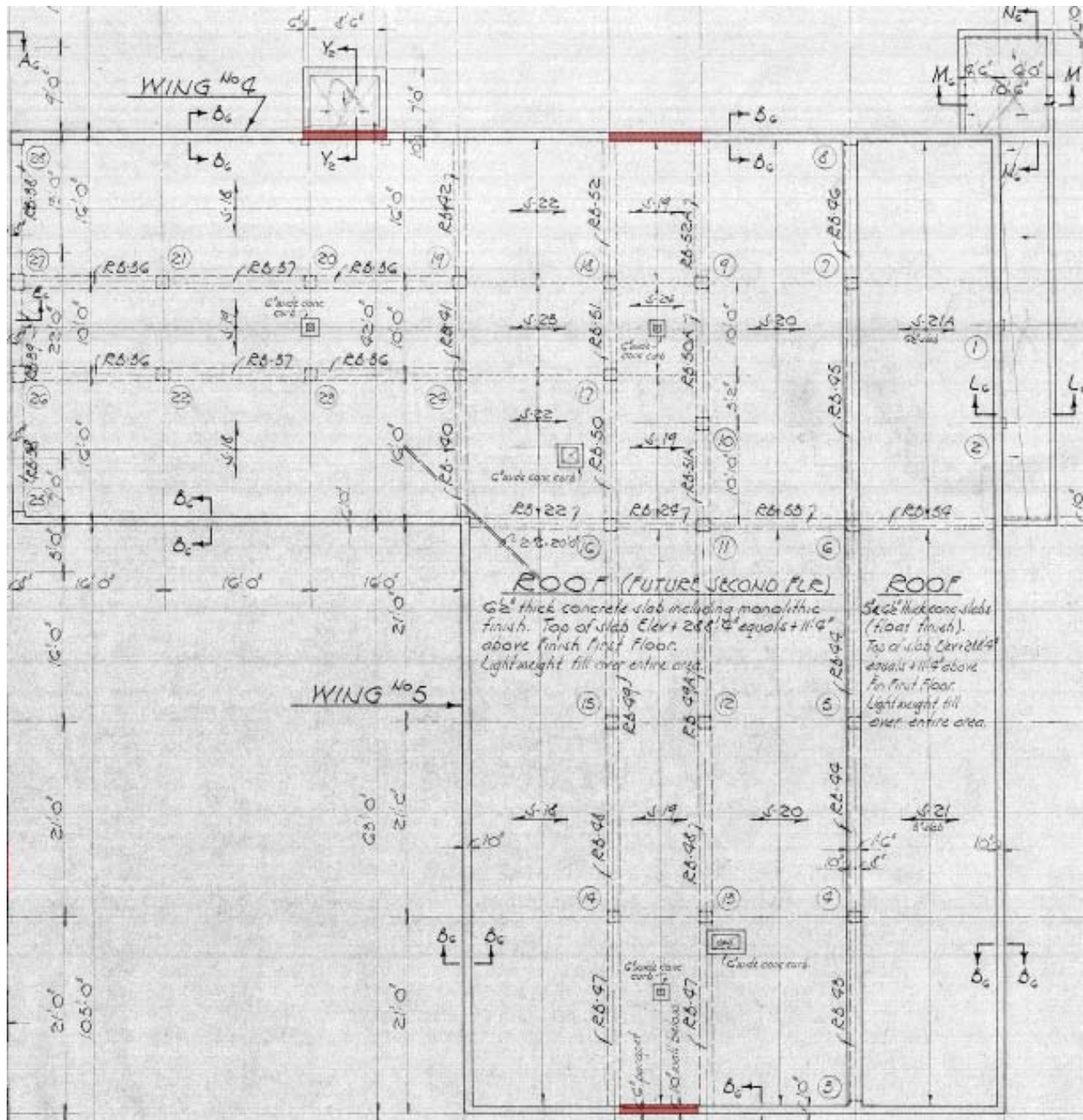
3325 Chanate Road, Santa Rosa, CA 95404

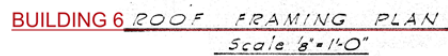
BUILDING 3 – JACKETED CONCRETE BEAM LOCATIONS (SHADED IN RED)

| 313



3325 Chanate Road, Santa Rosa, CA 95404

BUILDING 5 – JACKETED CONCRETE BEAM LOCATIONS (SHADED IN RED)

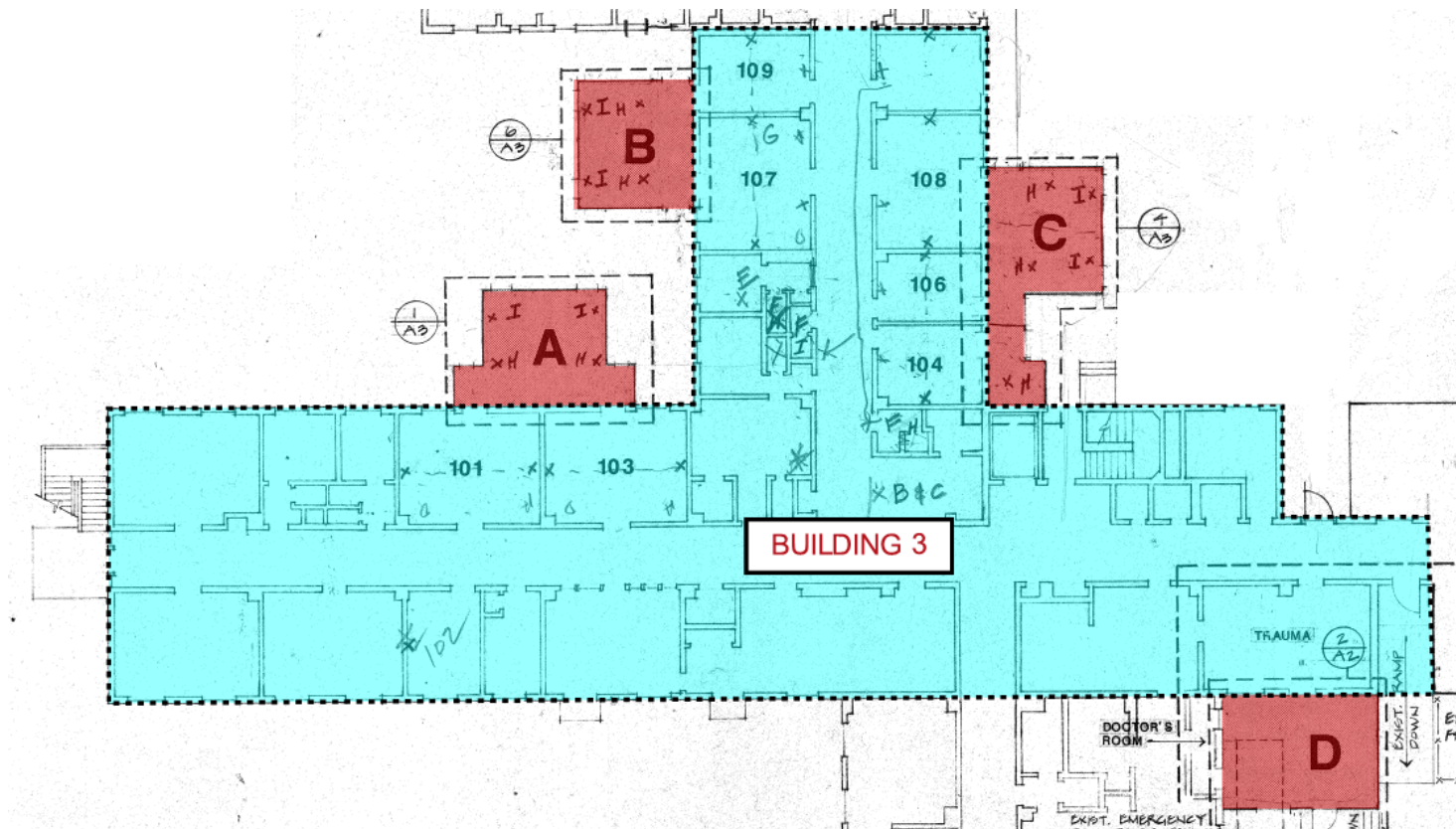


NTS



NTS

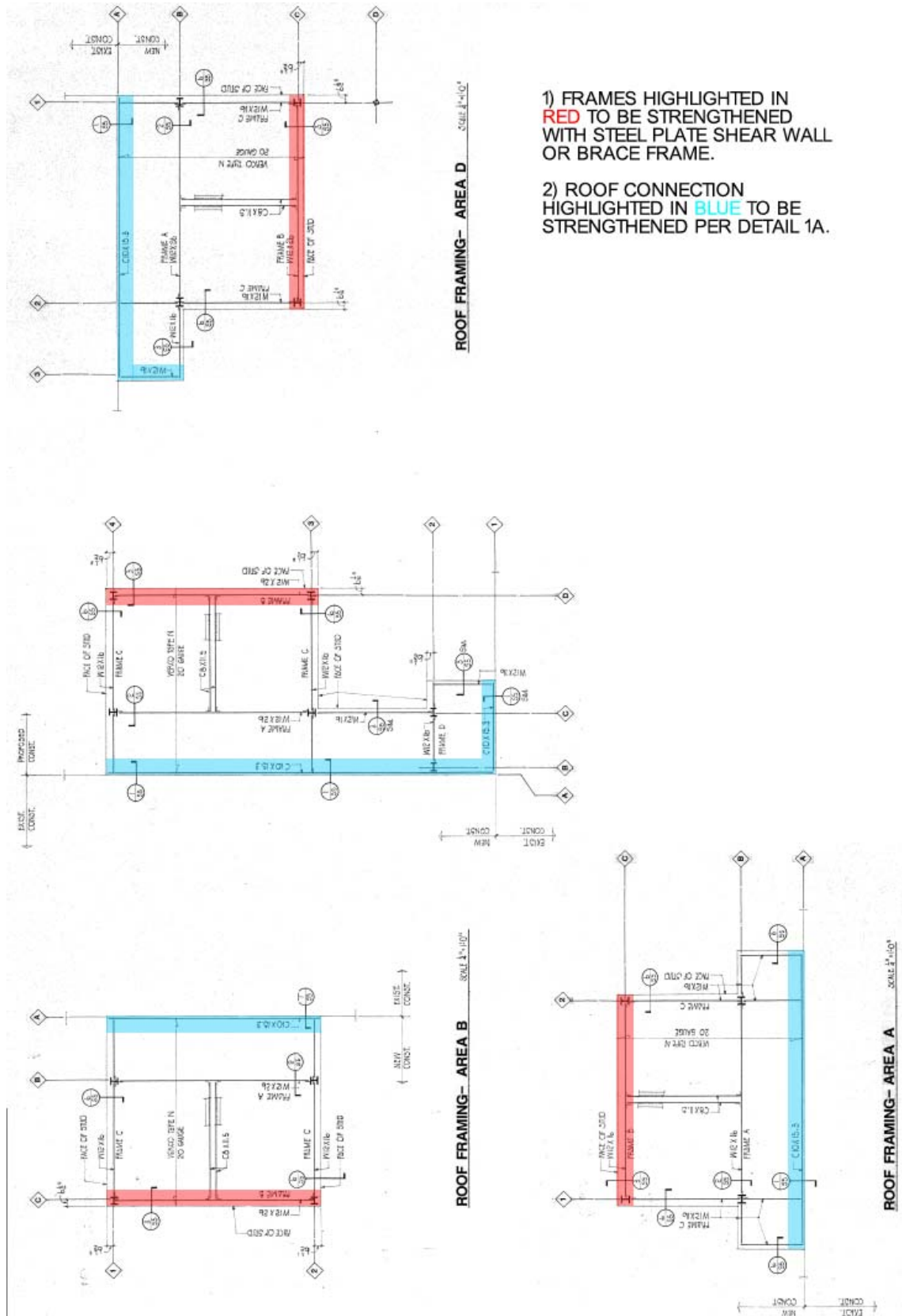
3325 Chanate Road, Santa Rosa, CA 95404

Buildings 3 Steel Appendages (1956 Hospital Wing)**Building 3 Overall Plan – Four (4) Steel Appendages (In Red)****STEEL APPENDAGE STRENGTHENING OPTIONS:**

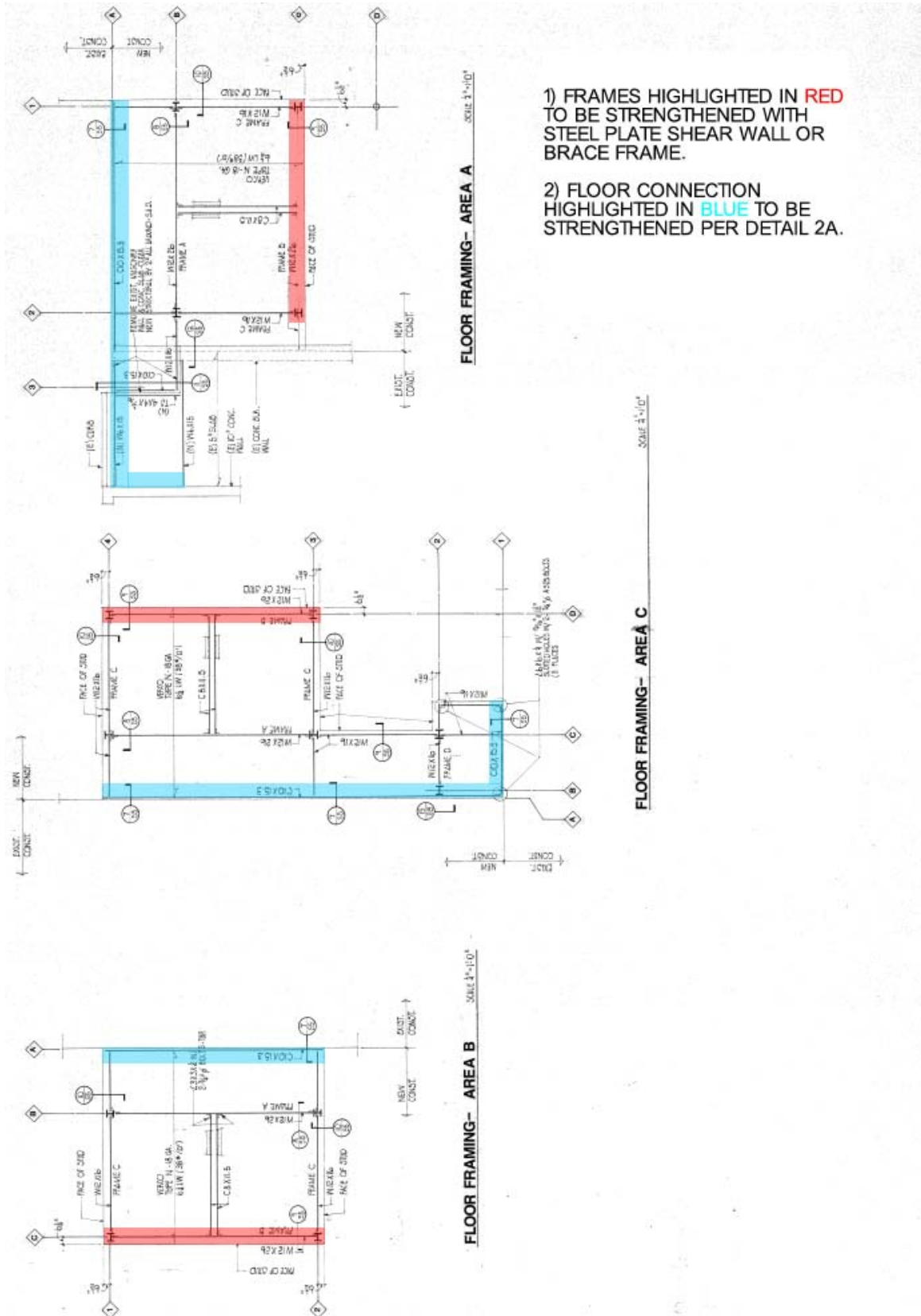
- A) NEW SHEAR WALL/BRACE FRAME + ADDITIONAL LEDGER ANCHORAGE*
- B) STRENGTHEN MOMENT FRAME MEMBERS + SEPARATE FLOORS/ROOFS FROM BUILDING 3*
- C) DEMOLISH THE FOUR (4) STEEL ADDITIONS

*SEE FOLLOWING SHEETS FOR SPECIFIC MARKUPS

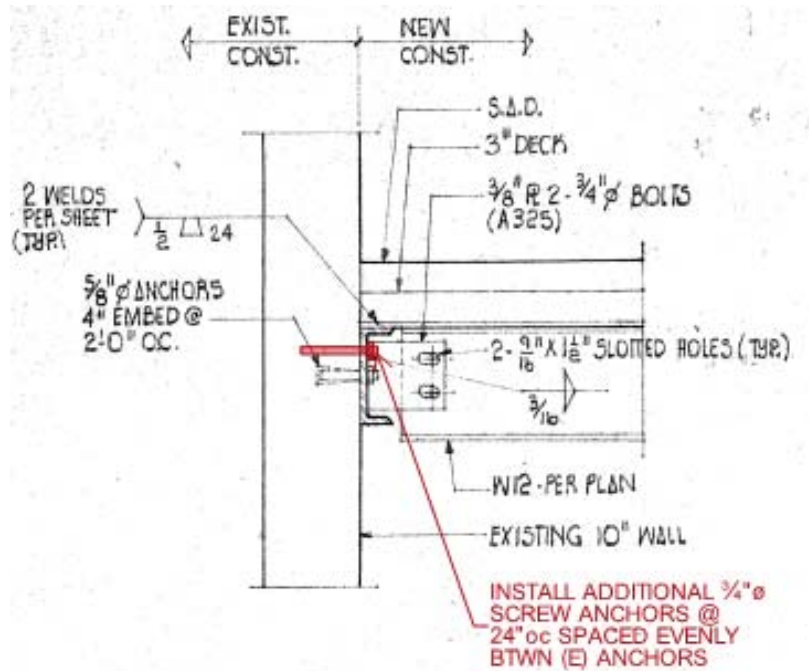
3325 Chanate Road, Santa Rosa, CA 95404

OPTION A: ROOF MARK-UPS

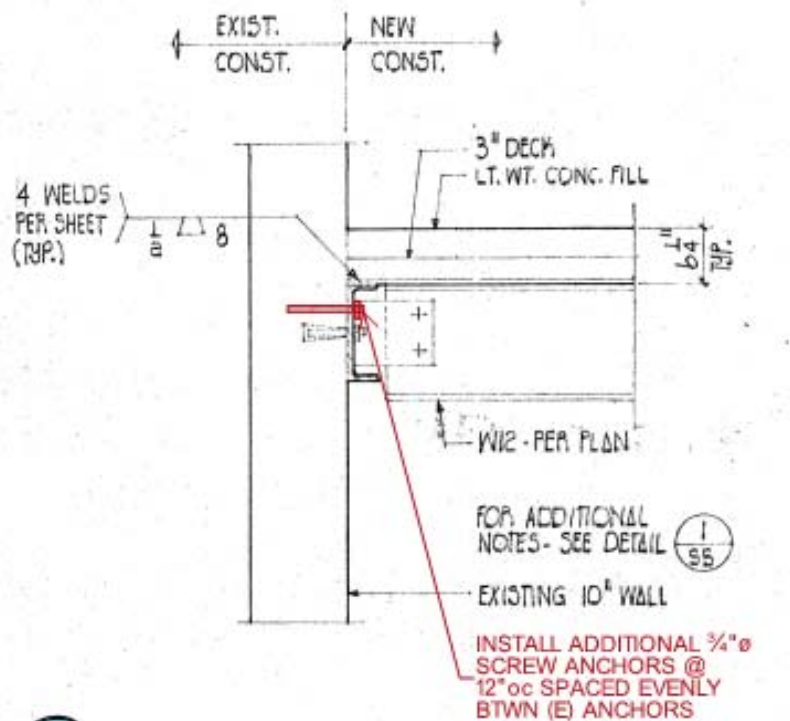
3325 Chanate Road, Santa Rosa, CA 95404

OPTION A: FLOOR MARK-UPS

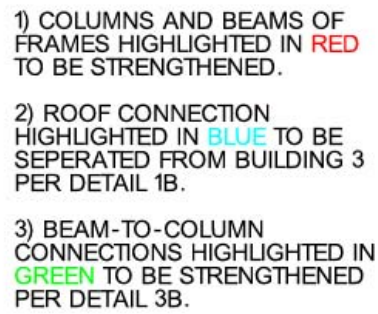
3325 Chanate Road, Santa Rosa, CA 95404

OPTION A: LEDGER CONNECTION MARK-UPS

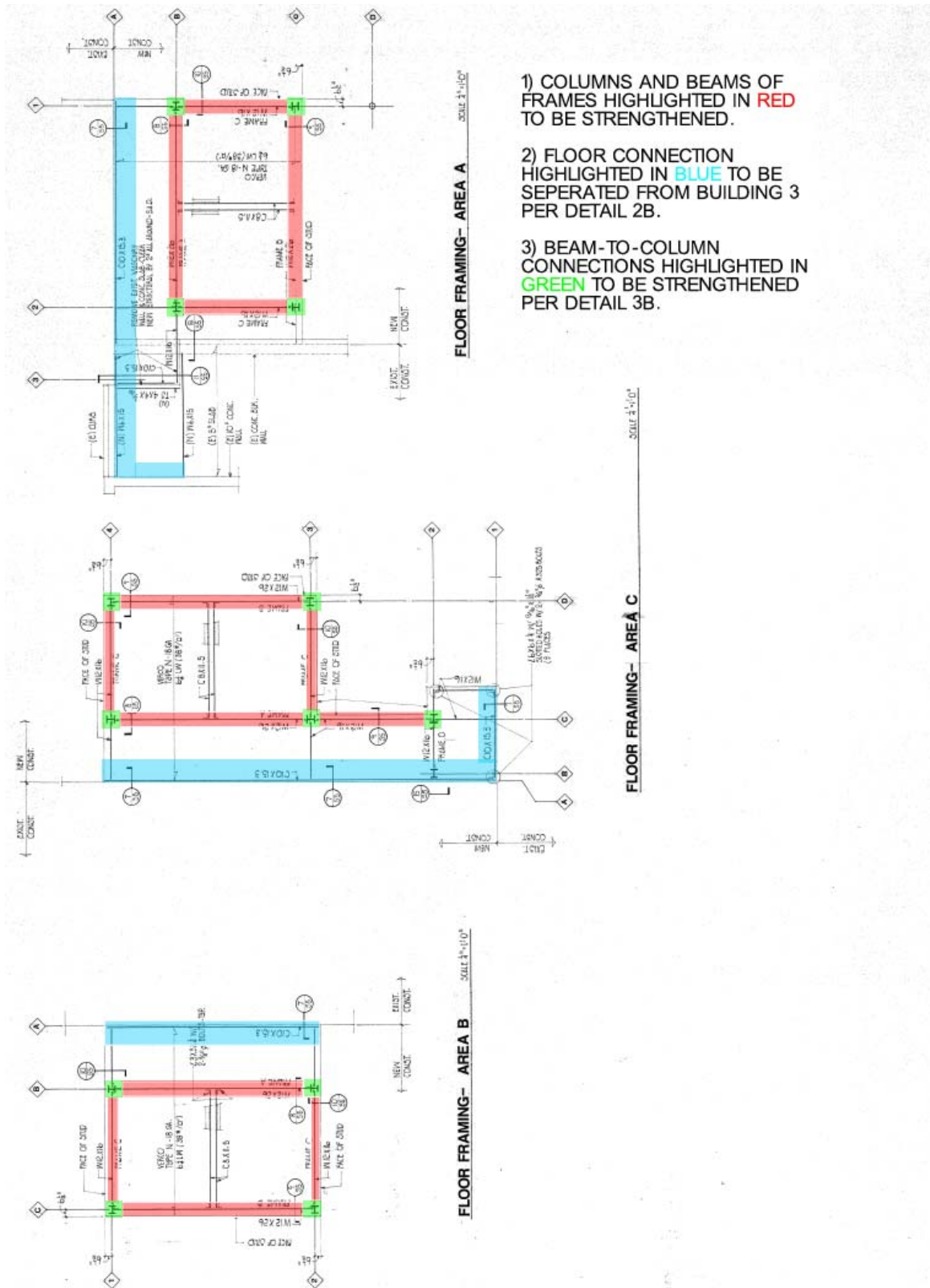
1A

SCALE $\frac{3}{4}$ " = 1'-0" U.O.N.

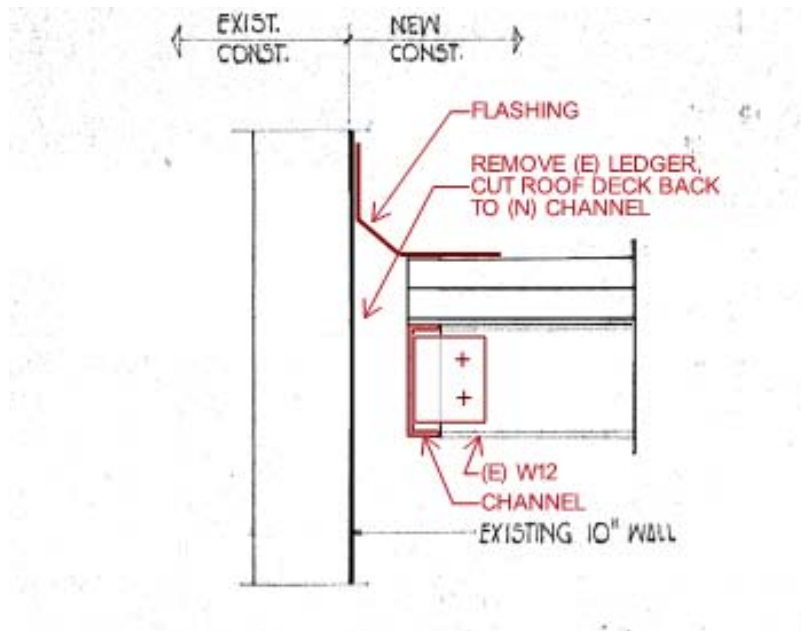
2A



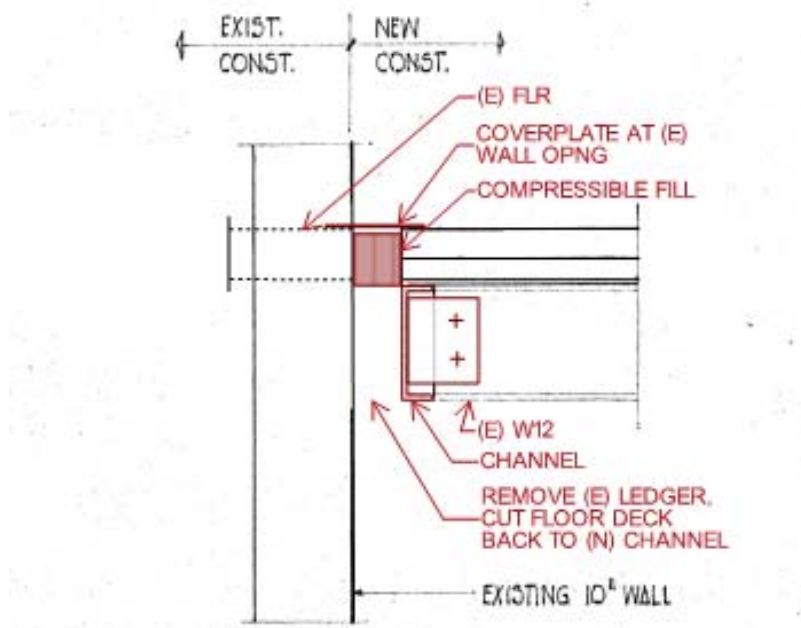
3325 Chanate Road, Santa Rosa, CA 95404

OPTION B: FLOOR MARK-UPS

3325 Chanate Road, Santa Rosa, CA 95404

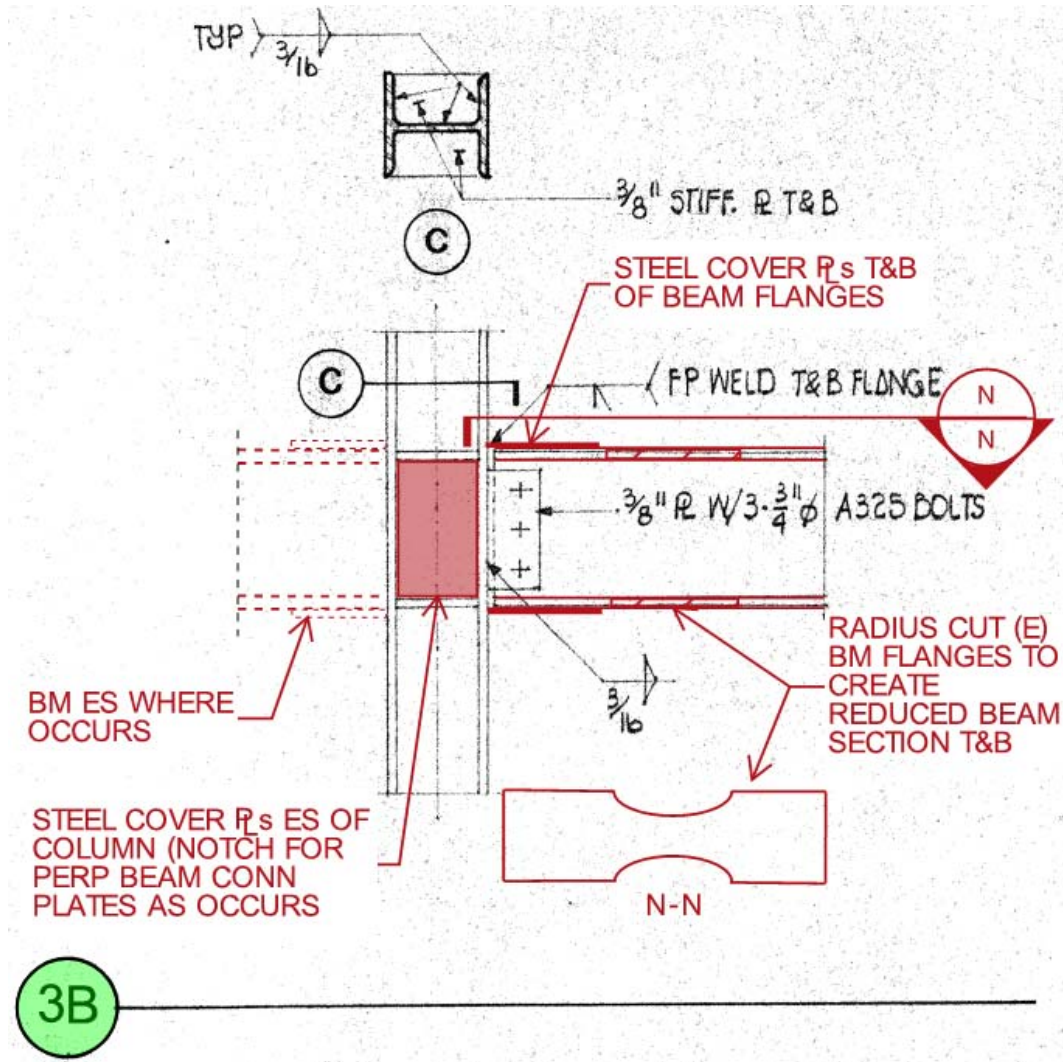
OPTION B: LEDGER CONNECTION MARK-UPS

1B SCALE $\frac{3}{4}" = 1'-0"$ U.O.N.



2B

OPTION B: FRAME BEAM – TO – COLUMN DETAIL MARK-UPS



3325 Chanate Road, Santa Rosa, CA 95404

Building 7 (1936 Original Hospital Building)

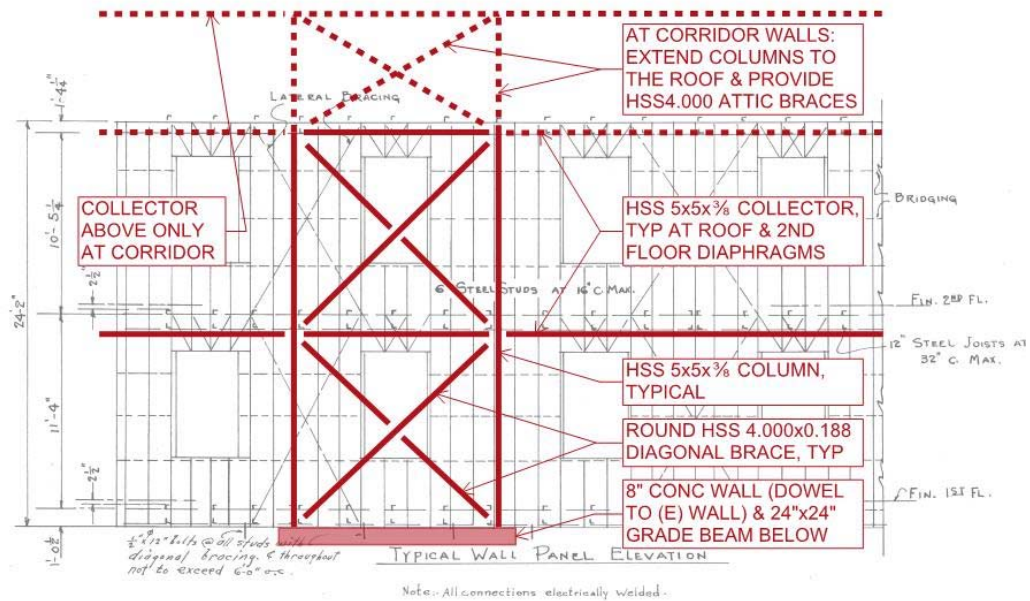
Schematic Strengthening Plan



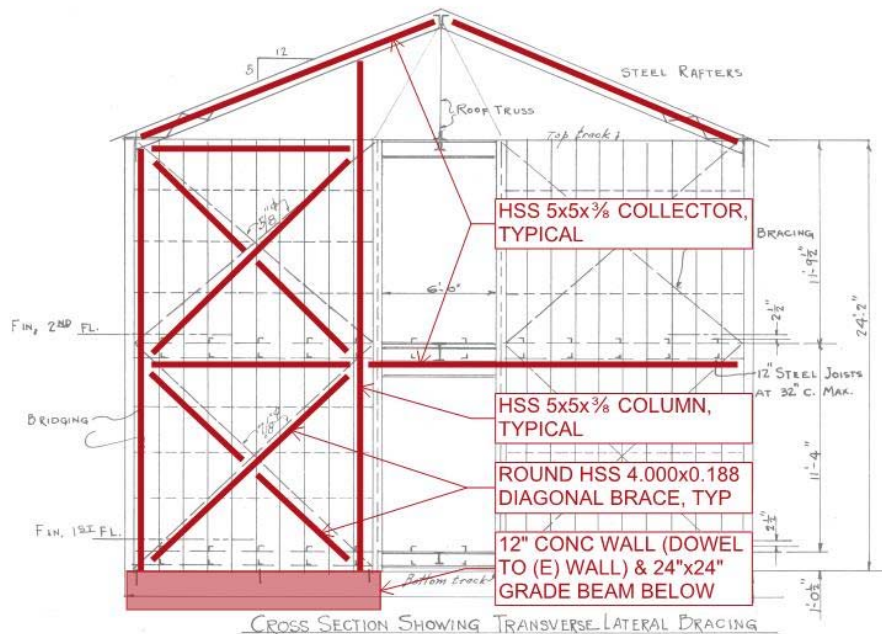
Sketch 1: Recommended new lateral force resisting system to be installed within the building.

3325 Chanate Road, Santa Rosa, CA 95404

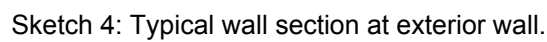
Schematic Strengthening Frame Elevations



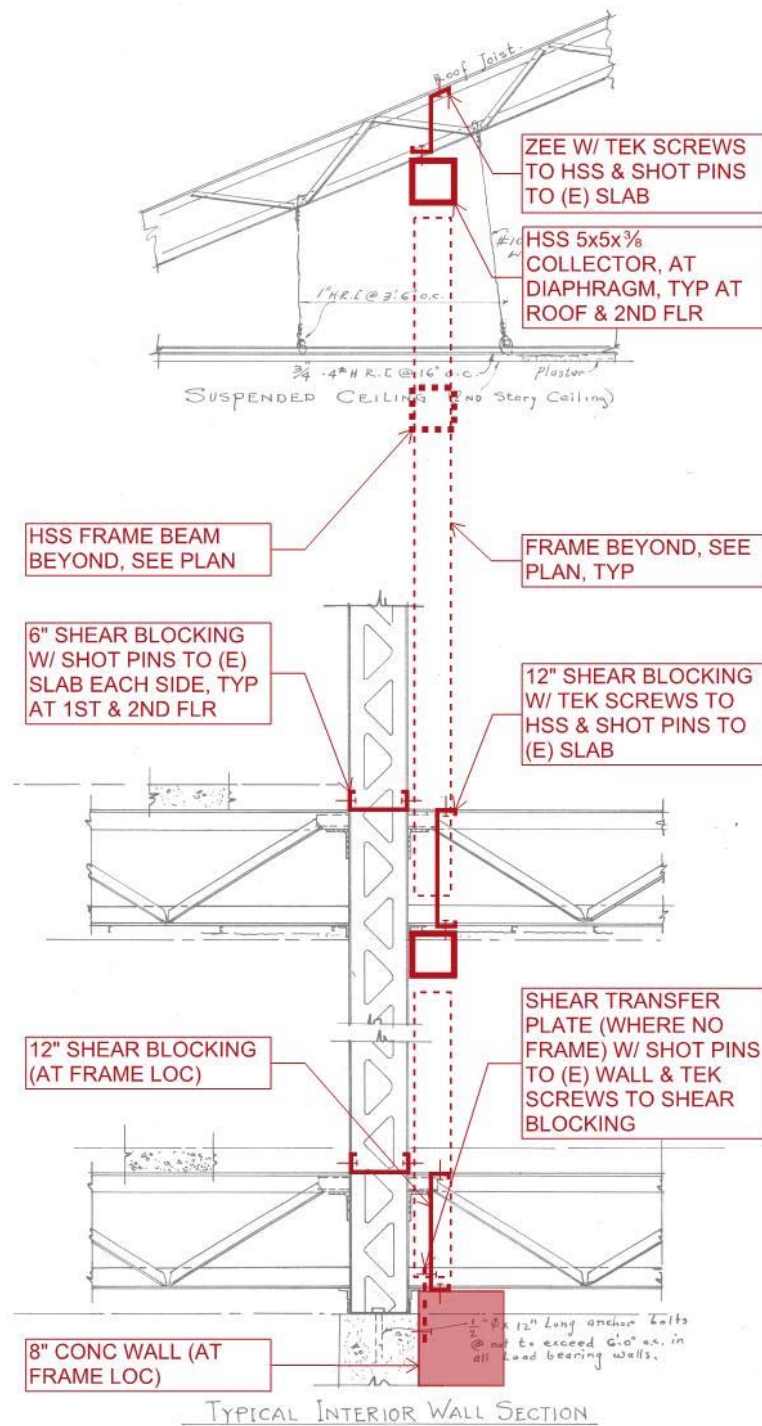
Sketch 2: Typical wall elevations at exterior and at corridor walls.



Sketch 3: Typical wall elevation at transverse walls.

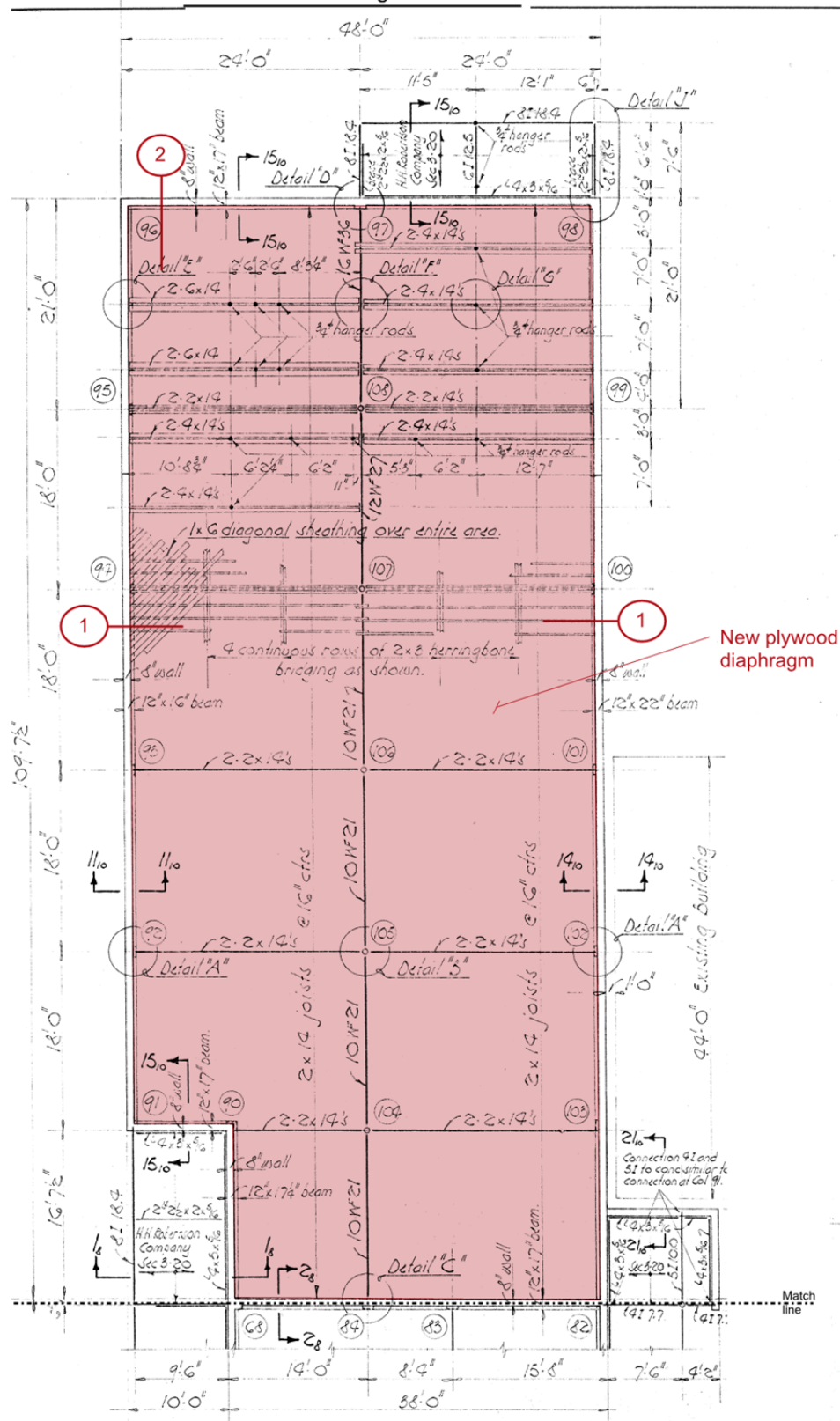


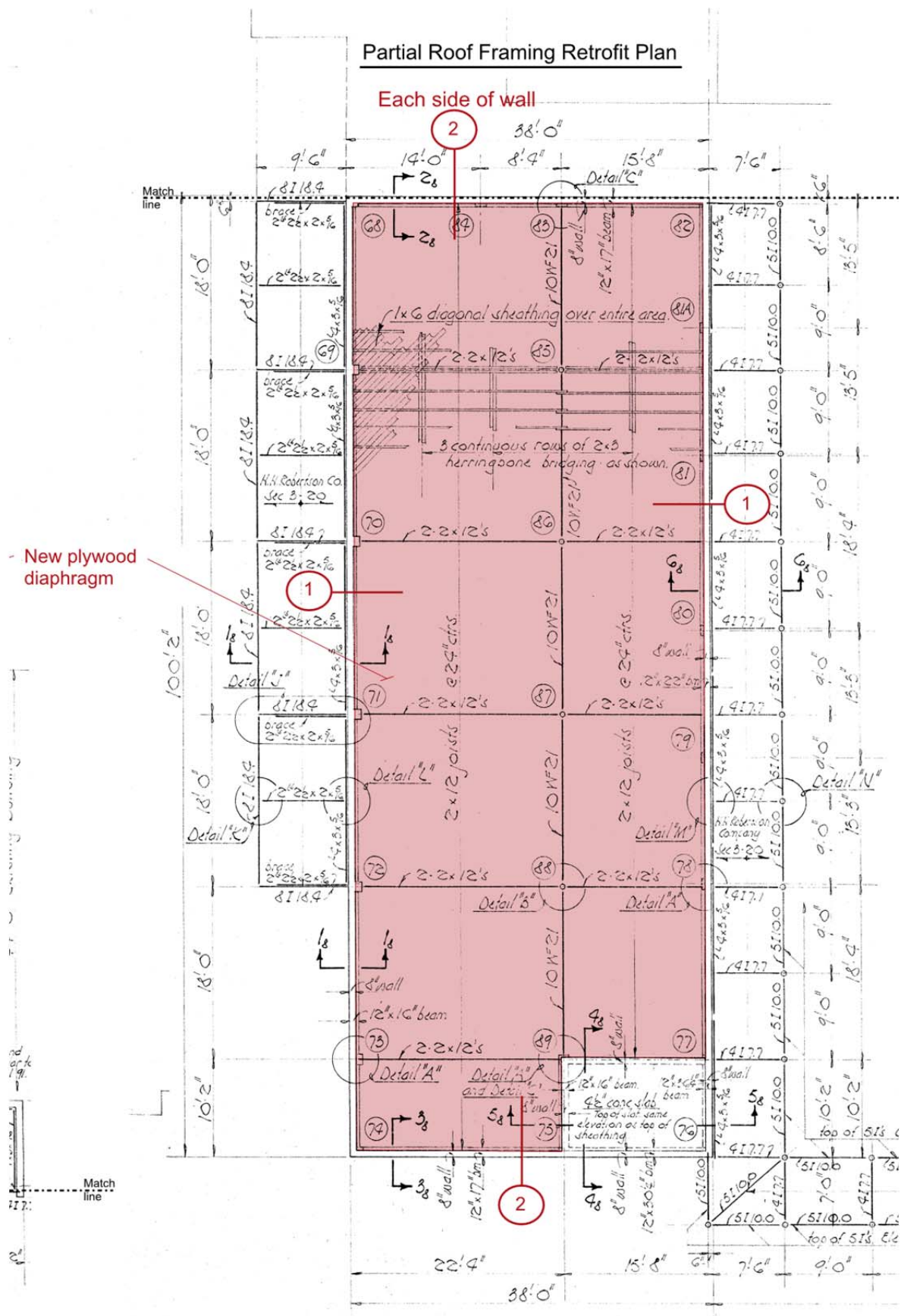
3325 Chanate Road, Santa Rosa, CA 95404



Sketch 5: Typical wall section at interior wall.

4



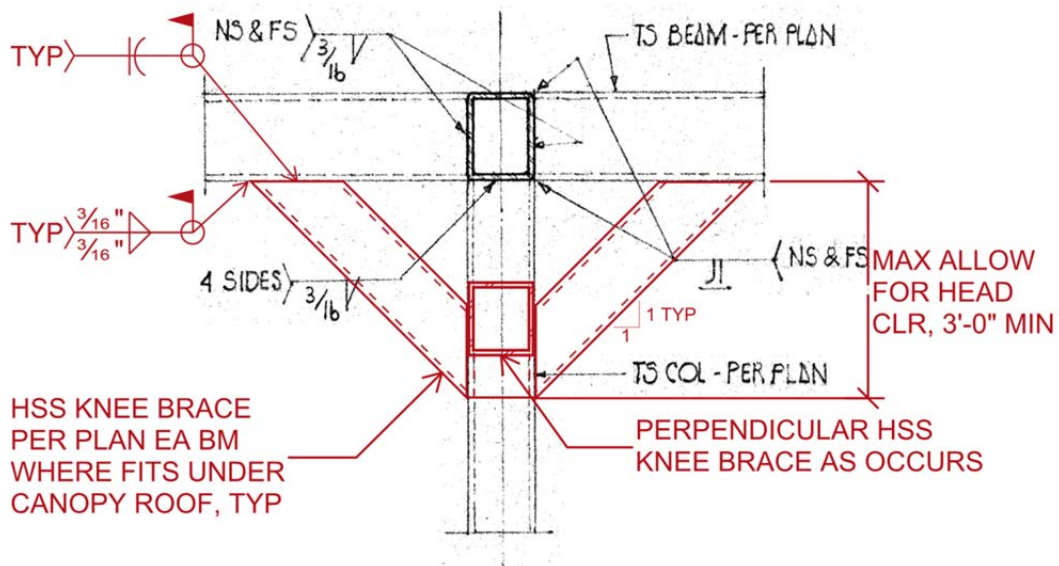


1

2

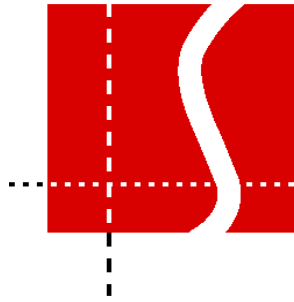
Retrofit Roof to Transverse Concrete Walls

Building 9 (1987 Ambulance Canopy)



NTS

APPENDIX H – CONSTRUCTION COST ESTIMATE



**Leland Saylor
Associates**
A Certified DVBE

SCHEMATIC ESTIMATE

SONOMA HOSPITAL COMPLEX

SANTA ROSA, CA

LSA JOB NUMBER:
14-132B

December 15, 2014

PREPARED FOR
ZFA STRUCTURAL ENGINEERS
BY LELAND SAYLOR ASSOCIATES



PROJECT: SONOMA HOSPITAL COMPLEX
LOCATION: SANTA ROSA, CA
CLIENT: ZFA STRUCTURAL ENGINEERS
DESCRIPTION: STRUCTURAL UPGRADES TO HOSPITAL COMPLEX

JOB NUMBER: 14-132B
PREPARED BY: JS
BID DATE: VARIOUS
ESTIMATE DATE: 12/15/2014

TABLE OF CONTENTS

SECTION	DESCRIPTION	PAGE
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II	SUMMARY OF THE ESTIMATE	11
III	BUILDING 1 - CATH LAB - FULL REPLACEMENT	13
IV	BUILDING 2 - ACUTE CARE HOSPITAL	22
V	BUILDINGS 3-6 - 1950'S & 60'S ADDITIONS	26
VI	BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2	31
VII	BUILDING 8 - KITCHEN/STORAGE	39
VIII	BUILDING 9 - AMBULANCE CANOPY	42



PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
CHECKED BY: **IS**
ESTIMATE DATE: **12/15/2014**

SECTION I

PREFACE AND NOTES TO THE ESTIMATE

PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
BID DATE: **VARIOUS**
ESTIMATE DATE: **12/15/2014**

PREFACE AND NOTES TO THE ESTIMATE

1.0 **PROJECT SYNOPSIS**

1.1 **TYPE OF STUDY:**

SCHEMATIC ESTIMATE

1.2 **PROJECT DESCRIPTION:**

Construction Type:	EXISTING
Foundation Type:	CONTINUOUS SPREAD FOOTINGS AS NEEDED
Exterior Wall Type:	PLASTER & OTHER
Roof Type:	NEW ON BUILDING 1
Stories Below Grade:	VARIOUS
Stories Above Grade:	VARIOUS
Sitework:	NONE
Plumbing System:	FIXTURE & PIPING REPLACEMENT AS NEEDED
Mechanical System:	REROUTING AS NEEDED
Fire Protection System:	REROUTING AS NEEDED
Electrical Service:	REROUTING & REPLACEMENT AS NEEDED

PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
BID DATE: **VARIOUS**
ESTIMATE DATE: **12/15/2014**

PREFACE AND NOTES TO THE ESTIMATE

1.3 GENERAL NOTES REGARDING PROJECT:

Structural Upgrades for multi-building hospital complex. Building one is a new building and has no structural upgrade recommendation except for full replacement in the case of ground failure. Bldg 2 is a column to beam strengthening project. Bldg 3-6 are a collection of additions that were built at various times. Bldg 7 is a full structural upgrade with repairs to (e) finishes disturbed by structural installations. Bldg 8 is a kitchen and storage complex, bldg 9 is a canopy.

2.0 DEFINITIONS

2.1 ESTIMATE OF COST:

An Estimate of Cost is prepared from a survey of the quantities of work - items prepared from written or drawn information provided at the design-development, working drawing or bid-documents stage of the design. Historical costs, information provided by contractors and suppliers, plus judgmental evaluation by the Estimator are used as appropriate as the basis for pricing. Allowances as appropriate will be included for items of work which are not indicated on the design documents provided that the Estimator is made aware of them, or which, in the judgment of the Estimator, are required for completion of the work. We cannot, however, be responsible for items or work of an unusual nature of which we have not been informed.

2.2 BID:

An offer to enter a contract to perform work for a fixed sum, to be completed within a limited period of time.

PROJECT: **SONOMA HOSPITAL COMPLEX**
 LOCATION: **SANTA ROSA, CA**
 CLIENT: **ZFA STRUCTURAL ENGINEERS**
 DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
 PREPARED BY: **JS**
 BID DATE: **VARIOUS**
 ESTIMATE DATE: **12/15/2014**

PREFACE AND NOTES TO THE ESTIMATE

3.0 **BIDS & CONTRACTS**

3.1 **MARKET CONDITIONS:**

In the current market conditions for construction, our experience shows the following results on competitive bids, as a differential from Leland Saylor Associates final estimates:

Number of Bids	Percentage Differential
1	+25 to 100%
2 - 3	+10 to 25%
4 - 5	0 to +10%
6 - 7	0 to -10%
8 or more	-10 to -20%

Accordingly, it is extremely important to ensure that a minimum of 4 to 5 valid bids are received. Since LSA has no control over the bid process, there is no guarantee that proposals, bids or construction cost will not vary from our opinions or our estimates. Please see Competitive Bidding Statement in the estimate detail section for more information.

PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
BID DATE: **VARIOUS**
ESTIMATE DATE: **12/15/2014**

PREFACE AND NOTES TO THE ESTIMATE

4.0 **ESTIMATE DOCUMENTS**

4.1 This Estimate has been compiled from the following documents and information supplied:

DRAWINGS:

Architectural
marked up (e)

Mechanical
None

Landscaping
None

Structural
marked up (e)

Plumbing
None

Accessibility Standards
None

Civil
None

Electrical
None

Other
None

SPECIFICATIONS / PROJECT MANUAL:

REPORTS

COSTS PROVIDED BY OTHERS:

NONE

4.2 The user is cautioned that significant changes in the scope of the project, or alterations to the project documents after completion of the schematic estimate can cause major cost changes. In these circumstances, Leland Saylor Associates should be notified and an appropriate adjustment made to the schematic estimate.

PROJECT: **SONOMA HOSPITAL COMPLEX**
 LOCATION: **SANTA ROSA, CA**
 CLIENT: **ZFA STRUCTURAL ENGINEERS**
 DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
 PREPARED BY: **JS**
 BID DATE: **VARIOUS**
 ESTIMATE DATE: **12/15/2014**

PREFACE AND NOTES TO THE ESTIMATE

5.0 GROSS SQUARE FEET

BUILDING	GSF
BUILDING 1 - CATH LAB - FULL REPLACEMENT	8,037
BUILDING 2 - ACUTE CARE HOSPITAL	56,181
BUILDINGS 3-6 - 1950'S & 60'S ADDITIONS	34,742
BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2	38,017
BUILDING 8 - KITCHEN/STORAGE	8,746
BUILDING 9 - AMBULANCE CANOPY	1,500
TOTAL GROSS SQUARE FEET	147,223

6.0 WAGE RATES

- 6.1 This Estimate is based on prevailing wage-rates and conditions currently applicable in SANTA ROSA, CA.

7.0 PRORATE ADDITIONS TO THE ESTIMATE

7.1 GENERAL CONDITIONS: 10.00%

An allowance based on 10.00% of the construction costs subtotal has been included for Contractor's General Conditions.

7.2 CONTINGENCY: 25.00%

An allowance based on 25.00% of the construction costs subtotal has been included for Design/Estimating Contingency.

NOTE: This allowance is intended to provide a Design Contingency sum only, for use during the design process. It is not intended to provide for a Construction Contingency sum.

PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
BID DATE: **VARIOUS**
ESTIMATE DATE: **12/15/2014**

PREFACE AND NOTES TO THE ESTIMATE

7.3 ESCALATION: 5.00%

An allowance of 5.00% has been included in this estimate for construction material & labor cost escalation up to the anticipated mid-point of construction, based on the following assumptions:

Construction start date:	VARIOUS
Construction period:	VARIOUS
Mid-point of construction:	VARIOUS - ALLOW 1 YEAR
Annual escalation rate:	5.00%
Allowance for escalation:	5.00%

ADDITIONAL TIME TO MID-POINTS OF CONSTRUCTIONS SHOULD BE ADDED AT 5% PER YEAR

No allowance has been made for Code Escalation or Technological Escalation.

7.4 BUILDING OCCUPANCY ADDER 20.00%

A building occupancy adder of 20.00% has been included in the prorates section of the estimate.

7.5 PHASING ALLOWANCE 10.00%

A Phasing Allowance of 10.00% has been included in the prorates section of the estimate.

7.6 BONDS: 2.00%

An allowance of 2.00% of the construction cost subtotal is included to provide for the cost of Payment and Performance Bonds, if required.

PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
BID DATE: **VARIOUS**
ESTIMATE DATE: **12/15/2014**

PREFACE AND NOTES TO THE ESTIMATE

7.7 CONTRACTOR'S FEE: 8.00%

An allowance based on 8.00% of the construction cost subtotal is included for Contractor's office Overhead and Profit has been included. Office overhead of the contractor is always included with the fee.

All field overhead of the contractor is included in the General Conditions section of the estimate.

8.0 SPECIAL NOTES PERTAINING TO THIS ESTIMATE

8.1 SPECIFIC INCLUSIONS:

The following items are specifically included in this estimate:

NONE (or list)

8.2 SPECIFIC EXCLUSIONS:

The following items are specifically excluded from this estimate:

HAZMAT
SOIL REMEDIATION



PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
CHECKED BY: **IS**
ESTIMATE DATE: **12/15/2014**

SECTION II

SUMMARY OF THE ESTIMATE

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: ESTIMATE SUMMARY

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 PROJECT GSF: 147,223

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
	BUILDING 1 - CATH LAB - FULL REPLACEMENT	8,037	SF	608.25	4,888,488
	BUILDING 2 - ACUTE CARE HOSPITAL	56,181	SF	50.52	2,838,477
	BUILDINGS 3-6 - 1950'S & 60'S ADDITIONS	34,742	SF	9.17	318,426
	BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2	38,017	SF	289.33	10,999,281
	BUILDING 8 - KITCHEN/STORAGE	8,746	SF	52.31	457,467
	BUILDING 9 - AMBULANCE CANOPY	1,500	SF	8.38	12,566
	TOTAL PROJECT COSTS			132.55	19,514,706
	PRORATES INCLUDED IN ABOVE COSTS				
	General Conditions	10.00%			
	Design Contingency	25.00%			
	Escalation - 1 YEAR ONLY	5.00%			
	Building occupancy adder	20.00%			
	Phasing Allowance	10.00%			
	Bonds	2.00%			
	Overhead and Profit	8.00%			



PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
CHECKED BY: **IS**
ESTIMATE DATE: **12/15/2014**

SECTION III

BUILDING 1 - CATH LAB - FULL REPLACEMENT

LELAND SAYLOR ASSOCIATES

PROJECT: **SONOMA HOSPITAL COMPLEX**
 LOCATION: **SANTA ROSA, CA**
 CLIENT: **ZFA STRUCTURAL ENGINEERS**
 DESCRIPTION: **BUILDING 1 - CATH LAB - FULL REPLACEMENT**

LSA JOB NO: **14-132B**
 PREPARED BY: **JS**
 CHECKED BY: **IS**
 ESTIMATE DATE: **12/15/2014**
 GSF: **8,037**

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
2.10	DEMOLITION			20.00	160,740
2.20	EXCAVATION, FILL & GRADING			5.94	47,776
2.50	SITE UTILITIES			45.00	361,665
2.60	GENERAL SITE WORK			14.61	117,421
	SITE SUBTOTAL			85.55	\$ 687,601
3.10	CONCRETE FOUNDATIONS			15.00	120,555
3.50	CONCRETE, SLABS ON GRADE			7.00	56,259
5.10	STRUCTURAL STEEL			11.00	88,407
5.50	MISC. IRON & ARCHITECTURAL METALS			1.00	8,037
6.10	CARPENTRY, ROUGH			15.00	120,555
6.20	CARPENTRY, FINISH			6.00	48,222
7.20	THERMAL & SOUND INSULATION			8.75	70,324
7.30	ROOFING & RIGID INSULATION			18.00	144,666
7.60	SHEET METAL & SKYLIGHTS			2.00	16,074
7.90	CAULKING & SEALANTS			2.00	16,074
8.10	HOLLOW METAL WORK			8.00	64,296
8.50	GLASS, GLAZING & SASH			12.00	96,444
8.70	FINISH HARDWARE			5.00	40,185
9.20	GYPSUM WALLBOARD, FURRING & STUDS			25.20	202,532
9.30	CERAMIC TILE			2.70	21,700
9.50	ACOUSTICAL TILE			5.40	43,400
9.70	RESILIENT FLOORS			6.80	54,652
9.80	PAINTING			6.55	52,642
10.15	TOILET PARTITIONS			1.52	12,200
10.40	TOILET ACCESSORIES			0.62	5,000
10.50	BUILDING SPECIALTIES, GENERAL			6.00	48,222
12.30	CABINETS			15.00	120,555
15.10	PLUMBING			30.00	241,110
15.30	HEATING, VENTILATING & AIR. COND.			45.00	361,665
15.55	FIRE PROTECTION			7.00	56,259
16.00	ELECTRICAL WORK			40.00	321,480
16.20	ELECTRICAL SPECIAL SYSTEMS			21.50	172,796
	SUBTOTAL BUILDING			324.04	\$ 2,604,310

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 1 - CATH LAB - FULL REPLACEMENT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 8,037

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
	TOTAL SITE AND BUILDING			409.59	\$ 3,291,911
	PRORATE DETAIL				
	General Conditions	10.00%			329,191
	Design Contingency	20.00%			658,382
	Escalation - 1 YEAR ONLY	5.00%			164,596
	Building occupancy adder	0.00%			-
	Phasing Allowance	0.00%			-
	SUBTOTAL			552.95	\$ 4,444,080
	Bonds	2.00%			88,882
	Overhead and Profit	8.00%			355,526
	TOTAL PROJECT COSTS			608.25	\$ 4,888,488

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 1 - CATH LAB - FULL REPLACEMENT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 8,037

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
1.10	GENERAL CONDITIONS SEE PRORATES ABOVE				
	SUBTOTAL 1.1				NONE
2.10	DEMOLITION DEMO (E) BUILDING	8,037	SF	20.00	160,740
	SUBTOTAL 2.1				160,740
2.20	EXCAVATION, FILL & GRADING EXCAVATION BACKFILL AND COMPACT GRADING FOR SITE, ALLOW	893 589 24,111	CY CY SF	25.00 35.00 0.20	22,325 20,628 4,822
	SUBTOTAL 2.2				47,776
2.50	SITE UTILITIES SEWER STORM DRAINS WATER FIRE WATER GAS TELEPHONE FIRE ALARM DATA	24,111 24,111 24,111 24,111 24,111 24,111 24,111 24,111	SF SF SF SF SF SF SF SF	1.50 2.00 1.75 2.50 1.75 1.75 1.75 2.00	36,167 48,222 42,194 60,278 42,194 42,194 42,194 48,222
	SUBTOTAL 2.5				361,665
2.60	GENERAL SITE WORK PAVING 66% OF SITE LANDSCAPING & IRRIGATION - 33% OF SITE FURNISHINGS	15,913 7,957 24,111	SF SF SF	5.00 4.00 0.25	79,566 31,827 6,028
	SUBTOTAL 2.6				117,421

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 1 - CATH LAB - FULL REPLACEMENT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 8,037

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
3.10	CONCRETE FOUNDATIONS FOUNDATIONS	8,037	SF	15.00	120,555
	SUBTOTAL 3.1				120,555
3.50	CONCRETE, SLABS ON GRADE SLAB ON GRADE, REBAR, FINISH	8,037	SF	7.00	56,259
	SUBTOTAL 3.5				56,259
3.60	REINFORCING INCL ABOVE				-
	SUBTOTAL 3.6				NONE
5.10	STRUCTURAL STEEL METAL DECK WITH CONCRETE FILL STRUCTURAL STEEL	8,037 8,037	SF SF	8.00 3.00	64,296 24,111
	SUBTOTAL 5.1				88,407
5.50	MISC. IRON & ARCHITECTURAL METALS MISC IRON	8,037	SF	1.00	8,037
	SUBTOTAL 5.5				8,037
6.10	CARPENTRY, ROUGH ROUGH CARPENTRY, WOOD FRAMED BUILDING	8,037	SF	15.00	120,555
	SUBTOTAL 6.1				120,555

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 1 - CATH LAB - FULL REPLACEMENT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 8,037

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
6.20	CARPENTRY, FINISH TRIMS AND OTHER FINISHES	8,037	SF	6.00	48,222
	SUBTOTAL 6.2				48,222
7.20	THERMAL & SOUND INSULATION ROOF INSULATION	8,037	SF	5.00	40,185
	WALL INSULATION INT & EXT	12,056	SF	2.50	30,139
	SUBTOTAL 7.2				70,324
7.30	ROOFING & RIGID INSULATION ROOF	8,037	SF	18.00	144,666
	SUBTOTAL 7.3				144,666
7.60	SHEET METAL & SKYLIGHTS SHEET METAL	8,037	SF	2.00	16,074
	SUBTOTAL 7.6				16,074
7.90	CAULKING & SEALANTS CAULKING & SEALANTS	8,037	SF	2.00	16,074
	SUBTOTAL 7.9				16,074
8.10	HOLLOW METAL WORK DOORS	8,037	SF	8.00	64,296
	SUBTOTAL 8.1				64,296
8.50	GLASS, GLAZING & SASH WINDOWS & STOREFRONT	8,037	SF	12.00	96,444
	SUBTOTAL 8.5, 8.8				96,444

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 1 - CATH LAB - FULL REPLACEMENT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 8,037

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
8.70	FINISH HARDWARE DOOR SETS	8,037	SF	5.00	40,185
	SUBTOTAL 8.7				40,185
9.20	GYPSUM WALLBOARD, FURRING & STUDS GYP WALLS & STUDS GYP CEILING	10,448 804	SF SF	18.00 18.00	188,066 14,467
	SUBTOTAL 9.2				202,532
9.30	CERAMIC TILE CERAMIC TILE FLOOR & WALLS	1,206	SF	18.00	21,700
	SUBTOTAL 9.3				21,700
9.50	ACOUSTICAL TILE EXPOSED GRID	7,233	SF	6.00	43,400
	SUBTOTAL 9.5				43,400
9.70	RESILIENT FLOORS FLOORING, SPECIALTY ANTI-STATIC IN MOST AREAS	6,831	SF	8.00	54,652
	SUBTOTAL 9.7				54,652
9.80	PAINTING PAINT INT WALLS GYP CEILINGS	20,896 8,037	SF SF	1.75 2.00	36,568 16,074
	SUBTOTAL 9.8				52,642

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 1 - CATH LAB - FULL REPLACEMENT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 8,037

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
10.15	TOILET PARTITIONS				
	PARTITIONS	8	EA	950.00	7,600
	ADA PARTITIONS	4	EA	1,150.00	4,600
	SUBTOTAL 10.15				12,200
10.40	TOILET ACCESSORIES				
	TOILET ACCESSORIES AND GRAB BARS	2	EA	2,500.00	5,000
	SUBTOTAL 10.4				5,000
10.50	BUILDING SPECIALTIES, GENERAL				
	BUILDING SPECIALTIES	8,037	SF	6.00	48,222
	SUBTOTAL 10.5				48,222
11.00	EQUIPMENT				
	EQUIPMENT - NIC				-
	SUBTOTAL 11.0				NONE
12.30	CABINETS				
	CASEWORK, ALLOW	8,037	SF	15.00	120,555
	SUBTOTAL 12.3				120,555
15.10	PLUMBING				
	PLUMING WORK	8,037	SF	30.00	241,110
	SUBTOTAL 15.1				241,110
15.30	HEATING, VENTILATING & AIR. COND.				
	HVAC WORK	8,037	SF	45.00	361,665
	SUBTOTAL 15.3				361,665

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 1 - CATH LAB - FULL REPLACEMENT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 8,037

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
15.55	FIRE PROTECTION FIRE PROTECTION	8,037	SF	7.00	56,259
	SUBTOTAL 15.55				56,259
16.00	ELECTRICAL WORK ELECTRICAL WORK	8,037	SF	40.00	321,480
	SUBTOTAL 16.0				321,480
16.20	ELECTRICAL SPECIAL SYSTEMS				
	TELEPHONE	8,037	SF	3.00	24,111
	FIRE ALARM	8,037	SF	6.50	52,241
	DATA	8,037	SF	6.00	48,222
	SECURITY	8,037	SF	3.00	24,111
	OTHER LOW VOLTAGE	8,037	SF	3.00	24,111
	SUBTOTAL 16.2				172,796



PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
CHECKED BY: **IS**
ESTIMATE DATE: **12/15/2014**

SECTION IV

BUILDING 2 - ACUTE CARE HOSPITAL

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 2 - ACUTE CARE HOSPITAL

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 56,181

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
2.10	DEMOLITION			1.14	64,000
	SITE SUBTOTAL			1.14	\$ 64,000
5.10	STRUCTURAL STEEL			4.21	236,352
9.10	LATH, PLASTER, FURRING & STUDS			10.25	576,000
9.20	GYPSUM WALLBOARD, FURRING & STUDS			6.84	384,000
9.80	PAINTING			1.56	87,709
	SUBTOTAL BUILDING			22.86	\$ 1,284,061
	TOTAL SITE AND BUILDING			23.99	\$ 1,348,061
	PRORATE DETAIL				
	General Conditions	18.00%			242,651
	Design Contingency	25.00%			337,015
	Escalation - 1 YEAR ONLY	5.00%			67,403
	Building occupancy adder	20.00%			269,612
	Phasing Allowance split floors	20.00%			269,612
	SUBTOTAL			45.11	\$ 2,534,355
	Bonds	2.00%			50,687
	Overhead and Profit	10.00%			253,435
	TOTAL PROJECT COSTS			50.52	\$ 2,838,477

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX			LSA JOB NO: 14-132B		
LOCATION: SANTA ROSA, CA			PREPARED BY: JS		
CLIENT: ZFA STRUCTURAL ENGINEERS			CHECKED BY: IS		
DESCRIPTION: BUILDING 2 - ACUTE CARE HOSPITAL			ESTIMATE DATE: 12/15/2014		
			GSF: 56,181		
SCHEMATIC ESTIMATE					
ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
1.10	GENERAL CONDITIONS - SEE PRORATES ABOVE				
	SUBTOTAL 1.1				NONE
2.10	DEMOLITION				
	DEMO FINISHES TO EXPOSE COLUMNS & BEAMS	112	EA	400.00	44,800
	DEMO FINISHES FOR BRACES	32	EA	600.00	19,200
	SUBTOTAL 2.1				64,000
5.10	STRUCTURAL STEEL				
	CUT DOGBONE (2 NOCHES) INTO (E) WF BEAM	112	EA	500.00	56,000
	STEEL PLATE @ SIDES OF BEAM, FULL LENGTH EXCEPT FOR DOGBONE CUTOUTS, ALLOW 9" X 10' X1/2"	17,136	LBS	3.50	59,976
	STEEL PLATE @ SIDES OF BEAM, BETWEEN DOGBONE CUTOUTS AND COLUMN, ALLOW 9" X 6" X3/4"	2,569	LBS	3.50	8,992
	TUBE STEEL BRACES, ALLOW 6X6X3/8	24,624	LBS	3.50	86,184
	GUSSET PLATES	7,200	LBS	3.50	25,200
	SUBTOTAL 5.1				236,352
9.10	LATH, PLASTER, FURRING & STUDS (incl. 5% laps & waste)				
	REPAIR EXTERIOR FINISHES TO BEAM-COLUMN CONNECTIONS	112	EA	3,600.00	403,200
	REPAIR FINISHES TO BRACE FRAME AREAS	32	EA	5,400.00	172,800
	SUBTOTAL 9.1				576,000

LELAND SAYLOR ASSOCIATES

PROJECT: **SONOMA HOSPITAL COMPLEX**
 LOCATION: **SANTA ROSA, CA**
 CLIENT: **ZFA STRUCTURAL ENGINEERS**
 DESCRIPTION: **BUILDING 2 - ACUTE CARE HOSPITAL**

LSA JOB NO: **14-132B**
 PREPARED BY: **JS**
 CHECKED BY: **IS**
 ESTIMATE DATE: **12/15/2014**
 GSF: **56,181**

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
9.20	GYPSUM WALLBOARD, FURRING & STUDS				
	REPAIR FINISHES TO BEAM-COLUMN CONNECTIONS	112	EA	2,400.00	268,800
	REPAIR FINISHES TO BRACE FRAME AREAS	32	EA	3,600.00	115,200
	SUBTOTAL 9.2				384,000
9.80	PAINTING				
	EXTERIOR:				
	PAINT EXTERIOR WALLS TO MATCH, 4-STORIES	17,803	SF	2.50	44,509
	INTERIOR:				
	PAINT AREAS OF REPAIR	144	EA	300.00	43,200
	SUBTOTAL 9.8				87,709



PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
CHECKED BY: **IS**
ESTIMATE DATE: **12/15/2014**

SECTION V

BUILDINGS 3-6 - 1950'S & 60'S ADDITIONS

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDINGS 3-6 - 1950'S & 60'S ADDITIONS

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 34,742

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
2.10	DEMOLITION			0.13	4,352
	SITE SUBTOTAL			0.13	\$ 4,352
3.20	CONCRETE, STRUCTURAL			1.14	39,594
3.50	CONCRETE, SLABS ON GRADE			0.17	6,000
5.10	STRUCTURAL STEEL			3.40	118,272
6.20	CARPENTRY, FINISH			0.03	1,200
9.20	GYPSUM WALLBOARD, FURRING & STUDS			0.02	864
	SUBTOTAL BUILDING			4.78	\$ 165,930
	TOTAL SITE AND BUILDING			4.90	\$ 170,282
	PRORATE DETAIL				
	General Conditions	10.00%			17,028
	Design Contingency	25.00%			42,570
	Escalation - 1 YEAR ONLY	5.00%			8,514
	Geographic Factor - Remote Site	20.00%			34,056
	Phasing Allowance	10.00%			17,028
	SUBTOTAL			8.33	\$ 289,479
	Bonds	2.00%			5,790
	Overhead and Profit	8.00%			23,158
	TOTAL PROJECT COSTS			9.17	\$ 318,426

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDINGS 3-6 - 1950'S & 60'S ADDITIONS

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 34,742

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
	ALTERNATES				
	OPTION B - A,B,C,D ADD'L STRENGTHENING				
	DELETE OPTION A WORK	(1)	LS	112,968	(112,968)
	DEMO ROOF & LEDGER @ CONNECTIONS	115	LF	16.00	1,840
	DEMO ADDITIONAL FINISHES	230	SF	2.00	460
	NEW ROOF AREA CONNECTIONS, NEW C CHANNEL	115	LF	95.00	10,925
	FLASHING	115	LF	20.00	2,300
	BEAM STRENGTHENING	445	LF	95.00	42,237
	CUT DOGBONE INTO (E) WF BEAM	112	EA	500.00	56,000
	STEEL PLATE @ TOP AND BOTTOM OF BEAM	56	EA	335.00	18,760
	STEEL COVER FOR EACH SIDE OF COLUMN-BEAM CONNECTION	56	EA	375.00	21,000
	PATCH FOR ADDITIONAL FINISHES	230	SF	13.00	2,990
	PRORATES				37,883
	TOTAL ADDER FOR OPTION B WITH PRORATES				81,427
	OPTION C - DEMO A,B,C,D INCL HAZMAT, PATCHING	1,515	SF	60.00	90,900
	DELETE OPTION A WORK	(1)	LS	112,968	(112,968)
	PRORATES				(19,199)
	TOTAL DEDUCTOR FOR OPTION C WITH PRORATES				(41,267)

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDINGS 3-6 - 1950'S & 60'S ADDITIONS

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 34,742

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
2.10	DEMOLITION REMOVE RUST FROM (E) STEEL L DEMO WALLS FOR CONCRETE WALL INSTALLATION, INTERIOR & EXTERIOR DEMO FLOORING AT WORK AREAS DEMO CEILING AT WORK AREAS	104 96 160 1,304	LF SF SF SF	10.00 4.00 2.00 2.00	1,040 384 320 2,608
	SUBTOTAL 2.1				4,352
3.20	CONCRETE, STRUCTURAL WALLS NEW 6" THICK CONCRETE SHEARWALLS DOWELS TO (E) WALLS DOWELS TO (E) FOUNDATIONS SLABS PATCH SLAB @ WALLS BEAMS AND GIRDERS FIBER-WRAP JACKETS	96 48 8 2 508	SF EA EA EA SFCA	50.00 125.00 125.00 1,200.00 50.00	4,800 6,000 1,000 2,400 25,394
	SUBTOTAL 3.2				39,594
3.50	CONCRETE, SLABS ON GRADE REPAIR SLABS AS NEEDED	1	LS	6,000.00	6,000
	SUBTOTAL 3.5				6,000
3.60	REINFORCING SEE CONCRETE SECTIONS				-
	SUBTOTAL 3.6				NONE

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDINGS 3-6 - 1950'S & 60'S ADDITIONS

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 34,742

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
5.10	STRUCTURAL STEEL				
	T.S. L	1,144	LBS	3.50	4,004
	3/4" ANCHOR BOLT	52	EA	25.00	1,300
	BRACE FRAMES, OPTION A	2,419	SF	40.00	96,768
	NEW ANCHOR BOLTS @ (E) MEMBERS OPTION A	216	EA	75.00	16,200
	SEE SUMMARY SHEET FOR OPTIONS B & C				
	SUBTOTAL 5.1				118,272



PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
CHECKED BY: **IS**
ESTIMATE DATE: **12/15/2014**

SECTION VI

BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2

LELAND SAYLOR ASSOCIATES

PROJECT: **SONOMA HOSPITAL COMPLEX**
 LOCATION: **SANTA ROSA, CA**
 CLIENT: **ZFA STRUCTURAL ENGINEERS**
 DESCRIPTION: **BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2
 RETROFIT**

LSA JOB NO: **14-132B**
 PREPARED BY: **JS**
 CHECKED BY: **IS**
 ESTIMATE DATE: **12/15/2014**
 GSF: **38,017**

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
2.10	DEMOLITION			15.22	578,656
2.20	EXCAVATION, FILL & GRADING			0.85	32,277
2.40	UNDERPINNING, SHORING & DEWATERING			4.66	177,024
	SITE SUBTOTAL			20.73	\$ 787,957
3.10	CONCRETE FOUNDATIONS			17.46	663,660
3.50	CONCRETE, SLABS ON GRADE			8.99	341,760
5.10	STRUCTURAL STEEL			34.61	1,315,694
5.50	MISC. IRON & ARCHITECTURAL METALS			2.00	76,034
6.10	CARPENTRY, ROUGH			7.19	273,408
6.20	CARPENTRY, FINISH			4.49	170,880
9.10	LATH, PLASTER, FURRING & STUDS			2.00	76,034
9.20	GYPSUM WALLBOARD, FURRING & STUDS			17.92	681,264
9.50	ACOUSTICAL TILE			3.60	136,704
9.70	RESILIENT FLOORS			2.92	111,072
9.80	PAINTING			4.38	166,644
12.30	CABINETS			2.10	80,000
15.10	PLUMBING			13.33	506,636
15.30	HEATING, VENTILATING & AIR. COND.			3.00	114,051
16.00	ELECTRICAL WORK			6.00	228,102
16.20	ELECTRICAL SPECIAL SYSTEMS			4.00	152,068
	SUBTOTAL BUILDING			133.99	\$ 5,094,011
	TOTAL SITE AND BUILDING			154.72	\$ 5,881,968

LELAND SAYLOR ASSOCIATES

PROJECT: **SONOMA HOSPITAL COMPLEX**
 LOCATION: **SANTA ROSA, CA**
 CLIENT: **ZFA STRUCTURAL ENGINEERS**
 DESCRIPTION: **BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2
 RETROFIT**

LSA JOB NO: **14-132B**
 PREPARED BY: **JS**
 CHECKED BY: **IS**
 ESTIMATE DATE: **12/15/2014**
 GSF: **38,017**

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
	PRORATE DETAIL				
	General Conditions	10.00%			588,197
	Design Contingency	25.00%			1,470,492
	Escalation - 1 YEAR ONLY	5.00%			294,098
	Building occupancy adder	20.00%			1,176,394
	Phasing Allowance	10.00%			588,197
	SUBTOTAL			263.02	\$ 9,999,346
	Bonds	2.00%			199,987
	Overhead and Profit	8.00%			799,948
	TOTAL PROJECT COSTS			289.33	\$ 10,999,281
	OPTION 1 - DEMO (E) BUILDING INCL HAZMAT	38,017	SF	30.00	1,140,510

LELAND SAYLOR ASSOCIATES

PROJECT: **SONOMA HOSPITAL COMPLEX**
 LOCATION: **SANTA ROSA, CA**
 CLIENT: **ZFA STRUCTURAL ENGINEERS**
 DESCRIPTION: **BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2
 RETROFIT**

LSA JOB NO: **14-132B**
 PREPARED BY: **JS**
 CHECKED BY: **IS**
 ESTIMATE DATE: **12/15/2014**
 GSF: **38,017**

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
2.10	DEMOLITION				
	SAWCUT SLAB	17,088	LF	10.00	170,880
	DEMO SLAB	17,088	SF	2.50	42,720
	DEMO WALLS FOR STRUCTURAL INSTALLATION	27,168	SF	3.00	81,504
	DEMO CEILING FOR STRUCTURAL INSTALLATION	17,088	SF	3.00	51,264
	DEMO FLOOR FRAMING FOR STRUCTURAL INSTALLATION	17,088	SF	3.00	51,264
	DEMO CABINETS, ALLOW	200	LF	20.00	4,000
	HAZMAT ABATEMENT FOR WALLS & CEILINGS	44,256	SF	4.00	177,024
	SUBTOTAL 2.1				578,656
2.20	EXCAVATION, FILL & GRADING				
	EXCAVATE FOR NEW FOUNDATIONS, HAND DIG	316	CY	95.00	30,062
	BACKFILL	63	CY	35.00	2,215
	SUBTOTAL 2.2				32,277
2.40	UNDERPINNING, SHORING & DEWATERING				
	SHORING, AS NEEDED	44,256	SF	4.00	177,024
	SUBTOTAL 2.4				177,024
3.10	CONCRETE FOUNDATIONS				
	NEW 24"X24" FOUNDATIONS, HAND PLACE, REBAR	42	CY	1,350.00	56,700
	DOWEL TO (E) FOUNDATION & SLAB, ALLOW 12" O.C.	8,544	EA	45.00	384,480
	REFURBISH CONCRETE ITEMS AS NECESSARY AND SEAL AS APPROPRIATE	4,272	LF	45.00	192,240
	12" CONCRETE WALL @ FOUNDATIONS	252	SF	65.00	16,380
	8" CONCRETE WALL @ FOUNDATIONS	252	SF	55.00	13,860
	SUBTOTAL 3.1				663,660

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2
 RETROFIT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 38,017

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
3.50	CONCRETE, SLABS ON GRADE				
	REPLACE SLABS, HAND PLACE	17,088	SF	20.00	341,760
	SUBTOTAL 3.5				341,760
3.60	REINFORCING				
	SEE CONCRETE SECTIONS				-
	SUBTOTAL 3.6				NONE
5.10	STRUCTURAL STEEL				
	TUBE STEEL COLUMNS, CUT-UP PIECING WITH BASE PLATES THROUGHOUT	40,713	LBS	4.25	173,032
	TUBE STEEL BRACES	21,000	LBS	3.50	73,500
	TUBE STEEL COLLECTORS	152,903	LBS	3.50	535,162
	12" STEEL SHEAR BLOCKING, SHOT PINS TO CONCRETE, TEK SCREWS TO TUBE STEEL	4,272	LF	95.00	405,840
	RENOVATE CORRODED STRUCTURAL MEMBERS AS APPROPRIATE	2,136	LF	60.00	128,160
	SUBTOTAL 5.1				1,315,694
5.50	MISC. IRON & ARCHITECTURAL METALS				
	MISC METALS, PLATES, CLIPS & ANGLES	38,017	SF	2.00	76,034
	SUBTOTAL 5.5				76,034
6.10	CARPENTRY, ROUGH				
	REPAIR FLOOR FRAMING	17,088	SF	8.00	136,704
	REPAIR CEILING FRAMING	17,088	SF	8.00	136,704
	SUBTOTAL 6.1				273,408

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2
 RETROFIT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 38,017

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
6.20	CARPENTRY, FINISH REPAIR TRIMS	8,544	LF	20.00	170,880
	SUBTOTAL 6.2				170,880
9.10	LATH, PLASTER, FURRING & STUDS (incl. 5% laps & waste) REPAIR PLASTER, ALLOW	38,017	SF	2.00	76,034
	SUBTOTAL 9.1				76,034
9.20	GYPSUM WALLBOARD, FURRING & STUDS NEW GYP & FURRING TO REPLACE (E)	75,696	SF	9.00	681,264
	SUBTOTAL 9.2				681,264
9.50	ACOUSTICAL TILE REPLACE CEILING FINISHES, ALLOW FOR MOSTLY ACT	17,088	SF	8.00	136,704
	SUBTOTAL 9.5				136,704
9.60	WOOD FLOORING				-
	SUBTOTAL 9.6				NONE
9.70	RESILIENT FLOORS ALLOW FOR FLOORING REPLACEMENT	17,088	SF	6.50	111,072
	SUBTOTAL 9.7				111,072

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2
 RETROFIT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 38,017

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
9.80	PAINTING				
	INTERIOR:				
	GYPBOARD WALLS	75,696	SF	1.75	132,468
	CEILING	17,088	SF	2.00	34,176
	SUBTOTAL 9.8				166,644
12.30	CABINETS				
	ALLOW FOR REPLACEMENT CABINETS	200	LF	400.00	80,000
	SUBTOTAL 12.3				80,000
15.10	PLUMBING				
	ALLOW FOR PLUMBING FIXTURE REINSTALL	50	EA	450.00	22,500
	ALLOW FOR RE-PIPING/NEW ROUGH INS	50	EA	3,600.00	180,000
	ALLOW FOR PLUMBING UTILITY REROUTING	38,017	SF	8.00	304,136
	SUBTOTAL 15.1				506,636
15.30	HEATING, VENTILATING & AIR. COND.				
	ALLOW FOR REROUTING DUCTS & EQUIPMENT REROUTING	38,017	SF	3.00	114,051
	SUBTOTAL 15.3				114,051
16.00	ELECTRICAL WORK				
	ALLOW FOR ELECTRICAL REROUTING & REPLACEMENT	38,017	SF	6.00	228,102
	SUBTOTAL 16.0				228,102

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX
 LOCATION: SANTA ROSA, CA
 CLIENT: ZFA STRUCTURAL ENGINEERS
 DESCRIPTION: BUILDING 7 - ORIGINAL HOSPITAL - OPTION 2
 RETROFIT

LSA JOB NO: 14-132B
 PREPARED BY: JS
 CHECKED BY: IS
 ESTIMATE DATE: 12/15/2014
 GSF: 38,017

SCHEMATIC ESTIMATE

ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
16.20	ELECTRICAL SPECIAL SYSTEMS				
	ALLOW FOR SPECIAL SYSTEMS REROUTING & REPLACEMENT	38,017	SF	4.00	152,068
	SUBTOTAL 16.2				152,068



PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
CHECKED BY: **IS**
ESTIMATE DATE: **12/15/2014**

SECTION VII

BUILDING 8 - KITCHEN/STORAGE

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX		LSA JOB NO: 14-132B			
LOCATION: SANTA ROSA, CA		PREPARED BY: JS			
CLIENT: ZFA STRUCTURAL ENGINEERS		CHECKED BY: IS			
DESCRIPTION: BUILDING 8 - KITCHEN/STORAGE		ESTIMATE DATE: 12/15/2014			
		GSF: 8,746			
SCHEMATIC ESTIMATE					
ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
2.10	DEMOLITION			2.50	21,865
	SITE SUBTOTAL			2.50	\$ 21,865
6.10	CARPENTRY, ROUGH			7.72	67,528
7.20	THERMAL & SOUND INSULATION			5.00	43,730
7.30	ROOFING & RIGID INSULATION			12.00	104,952
7.60	SHEETMETAL & SKYLIGHTS			0.75	6,560
	SUBTOTAL BUILDING			25.47	\$ 222,770
	TOTAL SITE AND BUILDING			27.97	\$ 244,635
	PRORATE DETAIL				
	General Conditions	10.00%			24,463
	Design Contingency	25.00%			61,159
	Escalation - 1 YEAR ONLY	5.00%			12,232
	Building occupancy adder	20.00%			48,927
	Phasing Allowance	10.00%			24,463
	SUBTOTAL			47.55	\$ 415,879
	Bonds	2.00%			8,318
	Overhead and Profit	8.00%			33,270
	TOTAL PROJECT COSTS			52.31	\$ 457,467

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX			LSA JOB NO: 14-132B		
LOCATION: SANTA ROSA, CA			PREPARED BY: JS		
CLIENT: ZFA STRUCTURAL ENGINEERS			CHECKED BY: IS		
DESCRIPTION: BUILDING 8 - KITCHEN/STORAGE			ESTIMATE DATE: 12/15/2014		
			GSF: 8,746		
SCHEMATIC ESTIMATE					
ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
2.10	DEMOLITION DEMO ROOF FOR WORK	8,746	SF	2.50	21,865
	SUBTOTAL 2.1				21,865
6.10	CARPENTRY, ROUGH NEW PLYWOOD DIAPHRAGM NEW BLOCKING ADDITIONAL NAILING AREAS	8,746 1,400 105	SF BF EA	5.50 9.00 65.00	48,103 12,600 6,825
	SUBTOTAL 6.1				67,528
7.20	THERMAL & SOUND INSULATION ROOF INSULATION	8,746	SF	5.00	43,730
	SUBTOTAL 7.2				43,730
7.30	ROOFING & RIGID INSULATION ROOFING	8,746	SF	12.00	104,952
	SUBTOTAL 7.3				104,952
7.60	SHEETMETAL & SKYLIGHTS SHEETMETALS FOR ROOF	8,746	SF	0.75	6,560
	SUBTOTAL 7.6				6,560



PROJECT: **SONOMA HOSPITAL COMPLEX**
LOCATION: **SANTA ROSA, CA**
CLIENT: **ZFA STRUCTURAL ENGINEERS**
DESCRIPTION: **STRUCTURAL UPGRADES TO HOSPITAL COMPLEX**

JOB NUMBER: **14-132B**
PREPARED BY: **JS**
CHECKED BY: **IS**
ESTIMATE DATE: **12/15/2014**

SECTION VIII

BUILDING 9 - AMBULANCE CANOPY

LELAND SAYLOR ASSOCIATES

PROJECT: SONOMA HOSPITAL COMPLEX			LSA JOB NO: 14-132B		
LOCATION: SANTA ROSA, CA			PREPARED BY: JS		
CLIENT: ZFA STRUCTURAL ENGINEERS			CHECKED BY: IS		
DESCRIPTION: BUILDING 9 - AMBULANCE CANOPY			ESTIMATE DATE: 12/15/2014		
			GSF: 1,500		
SCHEMATIC ESTIMATE					
ITEM #	DESCRIPTION	QUANTITY	UNIT	COST	TOTAL
5.10	STRUCTURAL STEEL			4.48	6,720
	SUBTOTAL BUILDING			4.48	\$ 6,720
	TOTAL SITE AND BUILDING			4.48	\$ 6,720
	PRORATE DETAIL				
	General Conditions	10.00%			672
	Design Contingency	25.00%			1,680
	Escalation - 1 YEAR ONLY	5.00%			336
	Building occupancy adder	20.00%			1,344
	Phasing Allowance	10.00%			672
	SUBTOTAL			7.62	\$ 11,424
	Bonds	2.00%			228
	Overhead and Profit	8.00%			914
	TOTAL PROJECT COSTS			8.38	\$ 12,566
5.10	STRUCTURAL STEEL				
	NEW T.S. KNEE BRACES	720	LBS	6.00	4,320
	PAINTING	12	EA	200.00	2,400
	SUBTOTAL 5.1				6,720